SRWRS

Sacramento River Water Reliability Study

Engineering Technical Report for the SRWRS Elverta Diversion Alternative

November 2006

SACRAMENTO RIVER WATER RELIABILITY STUDY

Engineering Technical Report for the SRWRS Elverta Diversion Alternative

TABLE OF CONTENTS

LIST OF APPENDICES

LIST OF TABLES

LIST OF FIGURES

LIST OF ACRONYMS AND ABBREVIATIONS

°C	degrees Celsius
ABFSHIP	American Basin Fish Screen and Habitat Improvements Project
amp	ampere
APDS	Auburn Placer Disposal Service
ASR	aquifer storage and recharge
ATPS	Auburn Tunnel Pump Station
BA	Biological Assessment
BAT	best available technology
$CaCO_3$	calcium carbonate
Cal-OSHA	California Occupational Safety and Health Administration
CDFG	California Department of Fish and Game
CDX	Central Data Exchange
CEQA	California Environmental Quality Act
cfs	cubic feet per second
CGS	California Geological Survey
CMP	Coordinated Monitoring Program
CMU	concrete masonry units
CPT	control power transformer
CSD-1	County Sanitation District – 1
CT	product of disinfection concentration and contact time
CVPIA	Central Valley Project Improvement Act
CVRWQCB	Central Valley Regional Water Quality Control Board
CWS	community water system
D/DBP	disinfectant/disinfection by-product
DBE	Design Basis Earthquake

DHS	California Department of Health Services
DOC	dissolved organic carbon
DWR	California Department of Water Resources
EIR	Environmental Impact Report
EIS	Environmental Impact Statement
elevation	elevation in feet above mean sea level
ESWTR	Enhanced Surface Water Treatment Rule
FAA	Federal Aviation Administration
FIRM	Flood Insurance Rate Maps
fps	foot per second
G	velocity gradient
GAC	granular activated carbon
gpm/sf	gallons per minute per square foot
GWUDIS	groundwater under the direct influence of surface water
HAA5	five haloacetic acids
HPC	heterotropic plate count
IDSE	Initial Distribution System Evaluation
$KMnO_4$	potassium permanganate
kV	kilovolt
kVA	kilovolt-ampere
L	liter
L/mg-m	liter per milligram-meter
$lb/DS/ft^2$	pounds dry solids per square foot
LRAA	locational running annual average
LT2ESWTR	Long-Term 2 Enhanced Surface Water Treatment Rule
M&I	municipal and industrial

MCL	maximum contaminant level
MCLG	maximum containment level goal
MG	million gallons
mg/L	milligram per liter
mgd	million gallons per day
mL	milliliter
mm	millimeter
MRAA	maximum running annual average
MRDL	maximum contaminant level goal
MRDLG	maximum residual disinfectant level goal
MRF	Materials Recovery Facility
MRL	minimum reporting limit
MSL	mean sea level
MTBE	methyl tertiary butyl ether
NEDC	Natomas East Drainage Canal
NEPA	National Environmental Policy Act
NGVD 1929	National Geodetic Vertical Datum of 1929
NOAA	National Marine Fisheries Service, National Oceanic and Atmospheric Administration
NMWC	Natomas Mutual Water Company
NOI	Notice of Intent
NPDES	National Pollutant Discharge Elimination System
NTNCWS	nontransient, noncommunity water system
NTU	nephelometric turbidity unit
OEHHA	Office of Environmental Health Hazard Assessment
oocyst/L	oocyst per liter
OSHA	Occupational Safety and Health Administration

PACl	polyaluminum chloride
PCACD	Placer County Air Control District
PCWA	Placer County Water Agency
PG&E	Pacific Gas and Electric Company
PGA	peak ground acceleration
Phase I Report	SRWRS Initial Alternatives Report, Appendix C
PHG	Public Health Goal
POU	Place of Use
PSHA	Probabilistic Seismic Hazard Analysis
PT	potential transformer
PW	public works
RAA	running annual average
Reclamation	United States Department of the Interior, Bureau of Reclamation
RM	river mile
Roseville	City of Roseville
RR	railroad
SAA	Streambed Alteration Agreement
Sac Co.	Sacramento County
Sacramento	City of Sacramento
SAFCA	Sacramento Area Flood Control Agency
SDWA	Safe Drinking Water Act
SMAQMD	Sacramento Metropolitan Air Quality Management District
SMF	Sacramento International Airport
SMUD	Sacramento Municipal Utility District
SRCSD	Sacramento Regional County Sanitation District
SRWRS	Sacramento River Water Reliability Study

SSWD	Sacramento Suburban Water District
SUVA	specific ultraviolet absorbance
SWPPP	Stormwater Pollution Prevention Plan
SWRCB	State Water Resources Control Board
SWTR	Surface Water Treatment Rule
TBM	tunnel-boring machine
TCR	Total Coliform Rule
TDS	total dissolved solids
The Reclamation Board	The Reclamation Board of the State of California
TOC	total organic carbon
TSS	total suspended solids
TT	treatment technique
TTHM	total tirhalomethanes
UBC	Uniform Building Code
UCMR	Unregulated Contaminant Monitoring Rule
UPRR	Union Pacific Railroad
USACE	United States Army Corps of Engineers
USCG	United States Coast Guard
USEPA	United States Environmental Protection Agency
USFWS	United States Fish and Wildlife Service
USGS	Unites States Geological Survey
UV	ultraviolet
WPWMA	Western Placer Waste Management Authority
WTP	water treatment plant
WQ	Water Quality
µg/L	microgram per liter

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CHAPTER 1 INTRODUCTION

The Sacramento River Water Reliability Study (SRWRS) is being conducted by the United States Department of the Interior, Bureau of Reclamation (Reclamation), with four local cost-sharing partners: Placer County Water Agency (PCWA), Sacramento Suburban Water District (SSWD), City of Roseville (Roseville), and City of Sacramento (Sacramento). The goal of the SRWRS is to develop a water supply plan that is consistent with the Water Forum Agreement objectives of pursuing a Sacramento River diversion to meet water supply needs of the Placer-Sacramento region and promote ecosystem preservation along the lower American River.

1.1. STUDY BACKGROUND

Five water supply alternatives were developed and presented in the SRWRS Initial Alternatives Report (March 2005).[1] The alternatives considered were the Elkhorn/Elverta Diversion Alternative (subsequently renamed SRWRS Elverta Diversion Alternative), Sankey Diversion Alternative, Feather River Diversion Alternative, American River Pump Station Alternative (subsequently renamed ARPS-Elverta Diversion Alternative), and the Folsom Dam Alternative. Of the five alternatives, the SRWRS Elverta Diversion Alternative and the ARPS-Elverta Diversion Alternative were retained for further consideration. The SRWRS Elverta Diversion Alternative is the subject of this report while the ARPS-Elverta Diversion Alternative is presented in a separate document.

This report presents the engineering refinement for the SRWRS Elverta Diversion Alternative, as described below (see **Figure 1-1**). The SRWRS Elverta Diversion Alternative and associated facility plan have been designed to accommodate the needs of the SRWRS cost-sharing partners. That is, the infrastructure plan of the SRWRS Elverta Diversion Alternative includes a raw water intake and pump station located on the Sacramento River with a total discharge capacity of 235 million gallons per day (mgd), raw water conveyance pipelines, a new joint water treatment plant (WTP) of the same capacity, and treated water conveyance pipelines to the connecting points(s) of each cost-sharing partner's existing water distribution system.

This report also presents the engineering refinement for a subalternative of the SRWRS Elverta Diversion Alternative known as the Joint SRWRS-American Basin Fish Screen and Habitat Improvements Project (ABFSHIP) Elverta Diversion Alternative (see **Figure 1-2**. ABFSHIP would consolidate five existing Sacramento River diversions of the Natomas Mutual Water Company (NMWC) and several local riparian water right holders into two diversions with positive barrier fish screens. ABFSHIP also would eliminate a dam at the mouth of the Natomas Cross Canal to benefit the environment and the Sacramento River fishery. The two diversions on the Sacramento River are located where the levee intersects Sankey Road and Elkhorn Boulevard, respectively. The development of ABFSHIP was delayed by its environmental review process, and NMWC is currently preparing an Environmental Impact Statement/Environmental Impact Report (EIS/EIR) for ABFSHIP through Reclamation (National Environmental Policy Act (NEPA) lead agency) and California Department of Fish and Game (CDFG, California Environmental Quality Act (CEQA) lead agency). The schedule for implementing the recommended project is subject to funding availability from the Central Valley Project Improvement Act (CVPIA) Fish Screen Program.

Under the Joint SRWRS-ABFSHIP Elverta Diversion Alternative, the proposed Sacramento River intake at Elverta Road (Elverta Intake) and associated facility plan have been designed to accommodate the needs of the SRWRS cost-sharing partners, and the needs of NMWC, as provided by the Elkhorn

[1] Reclamation. 2005. Sacramento River Water Reliability Study Initial Alternatives Report. March.

Diversion planned in ABFSHIP. Also under this subalternative, NMWC would not construct the Elkhorn Diversion planned in ABFSHIP; instead, the proposed Elverta Intake would be expanded to include NMWC's required diversion capacity of 210 cubic feet per second (cfs) (135 mgd). The other key difference from the SRWRS Elverta Diversion Alternative is inclusion of improvements to approximately 1.6 miles of NMWC's existing Elkhorn Main Canal to allow delivery of raw water from the new Elverta Intake to NMWC facilities both north and south of the intake site.

1.2. OBJECTIVE OF THE REPORT

The primary objective of this report is to refine the engineering of key elements of the SRWRS Elverta Diversion Alternative and the Joint SRWRS-ABFSHIP Elverta Diversion Alternative to develop a project that can be evaluated as part of the environmental documentation process, including the Biological Assessment (BA) and the EIS/EIR. Engineering refinement of these alternatives includes completing feasibility-level engineering design to generate facility type and sizing requirements, site layouts, pipeline alignments, and related facility plans for power, sewage, and storm drainage, and identify proposed operating and construction characteristics.

1.3. ORGANIZATION OF THE REPORT

This report consists of eight chapters that present various aspects of the engineering refinement. Below is a brief discussion of each chapter.

Chapter 1 – Presents an introduction to the report, including study background, report objective, and report organization.

Chapter 2 – Presents a geotechnical characterization of the project areas and highlights potential hazards. Using the characterization, construction considerations are summarized and future geotechnical investigation recommendations are made.

Chapter 3 – Discusses the new Elverta Intake Facility, including design requirements, site selection, and river hydrology. Using this information, a preferred intake configuration is defined for the SRWRS Elverta Diversion Alternative. New and modified facilities required for the Joint SRWRS-ABFSHIP Elverta Diversion Alternative are also presented.

Chapter 4 – Discusses the raw water pipelines for the alternatives, including hydraulics, alignments, pipe materials, and pipeline appurtenances.

Chapter 5 – Discusses the new WTP (North Natomas WTP), including treated water goals and objectives for the project, regulatory requirements, and water quality evaluations. Using this information, the North Natomas WTP process selection and an overall facility design are presented.

Chapter 6 – Discusses the treated water pipelines for the alternatives, including hydraulics, alignments, special crossings, pipe materials, the PCWA booster pump station, pipeline appurtenances, and construction and operating characteristics.

Chapter 7 – Presents the construction cost estimate for all water supply components of the SRWRS Elverta Diversion Alternative and the Joint SRWRS-ABFSHIP Elverta Diversion Alternative.

Chapter 8 – Summarizes the regulatory requirements for constructing facilities as part of the Alternatives. This includes describing the permits that must be obtained and the recommended timing of activities related to obtaining the permits.

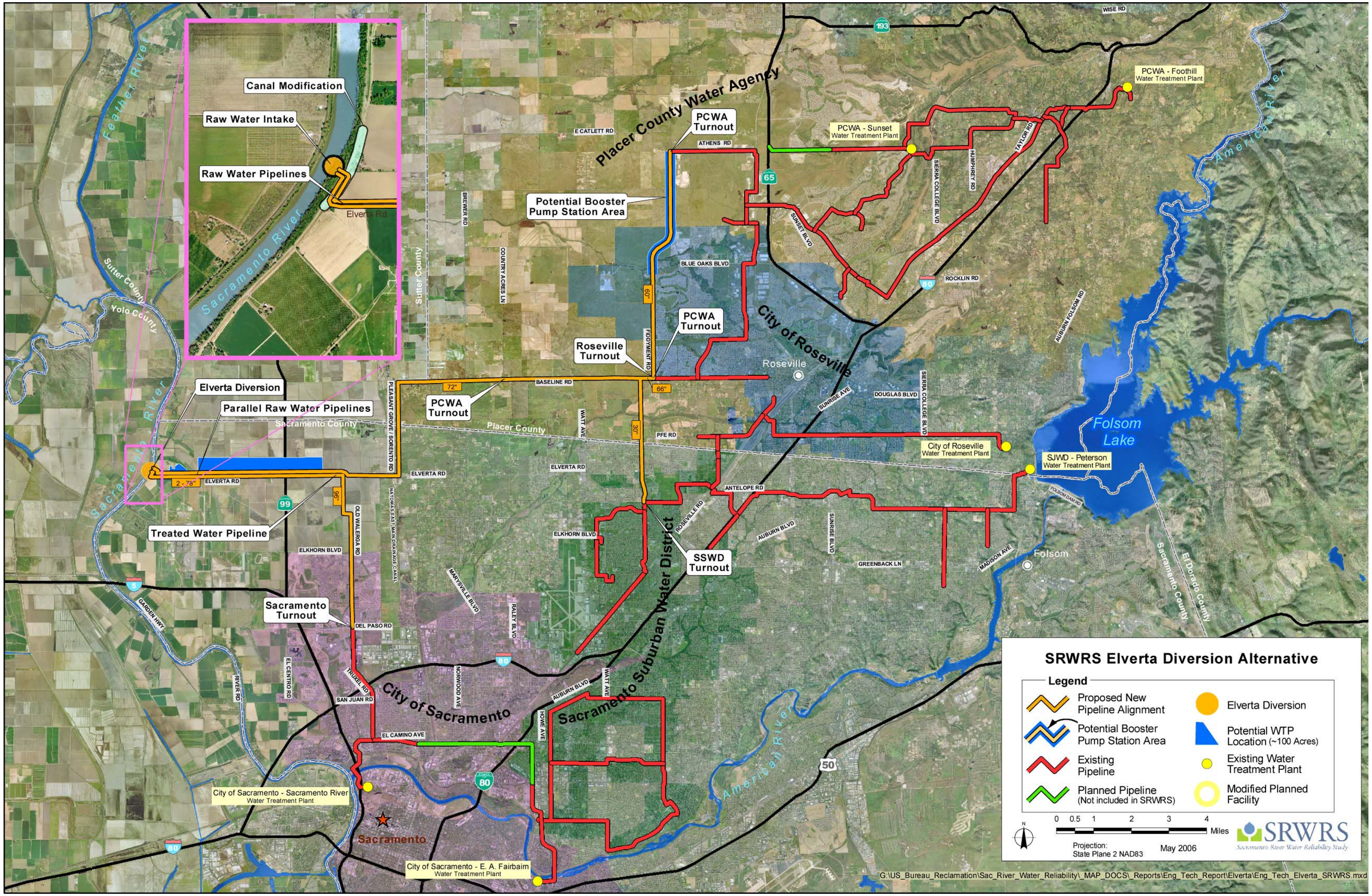

Figure 1-1 SRWRS Elverta Diversion Alternative

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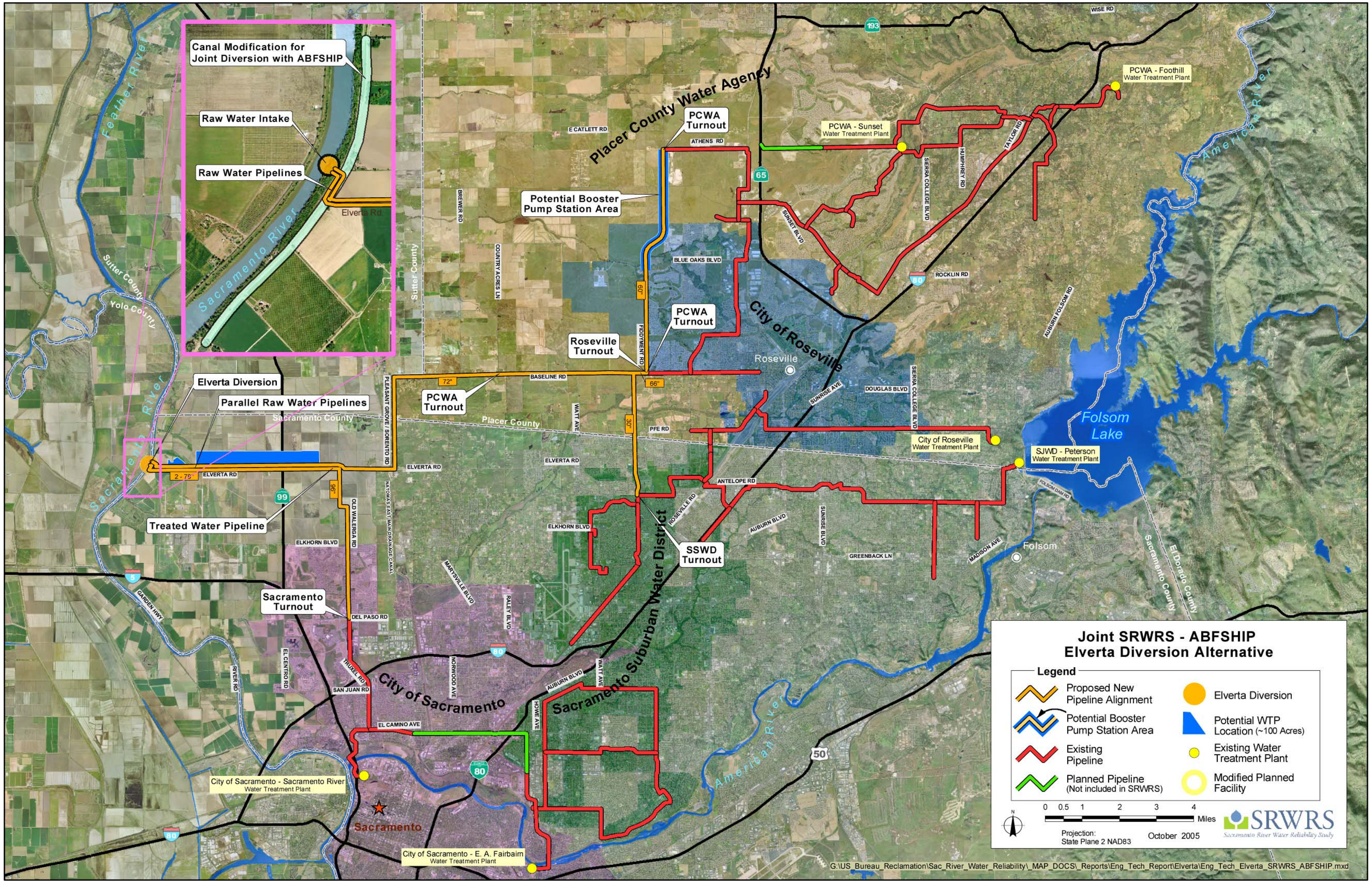

Figure 1-2 Joint SRWRS-ABFSHIP Elverta Diversion Alternative

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CHAPTER 2 GEOTECHNICAL CONDITIONS

This chapter on geotechnical conditions is intended to support feasibility-level design and cost estimates of the SRWRS Elverta Diversion Alternative and the Joint SRWRS-ABFSHIP Elverta Diversion Alternative for inclusion in the SRWRS. From a geotechnical perspective, the two alternatives are nearly identical and will be discussed as a single alternative, in this chapter except where noted. The chapter briefly describes the alternatives, and the location and nature of each component of the alternatives; presents regional geology and seismicity; details geotechnical conditions for the features associated with the alternatives; considers construction issues; discusses potential geotechnical hazards; and recommends future geotechnical investigations.

The alternative would be constructed in the Great Valley Geomorphological Province, which was formed by low-lying, flat topography that is characterized by Quaternary clay, silt, sand, and gravel deposited by flooding of the Sacramento River. Toward the east, these sediments overlap onto older alluvial fan deposits emanating from the Sierra Nevada foothills. Excavations for the intake, WTP, and pipelines in the western part of this province will have to contend with high groundwater levels. Excavations therefore would likely have to be dewatered and shored. The pipelines will cross several highways and canals, some of which would have to be tunneled.

Geologic hazards to the project to be evaluated in future studies include potentially liquefiable and corrosive soils. However, no geotechnical conditions appear to render the planned projects infeasible.

2.1. PROJECT DESCRIPTION

This section describes the study area and components of the alternatives.

2.1.1. Study Area

The study area can be defined by a triangle approximately 19 miles on each side, oriented in a westerly direction, with its apex at the intake on the Sacramento River. The study area is characterized by the flat topography of the Sacramento Valley.

2.1.2. Components of the Alternatives

The proposed alternatives (see **Figures 1-1** and **1-2**) consist of a new raw water intake, 235 mgd pump station (371 mgd for the Joint SRWRS-ABFSHIP Elverta Diversion Alternative), and access bridge that would be located on the left levee (east bank) of the Sacramento River near the intersection of Elverta Road and the Garden Highway, which runs along the top of the levee. Twin 66-inch-diameter raw water transmission pipelines (and an additional 72-inch-diameter pipeline for the Joint SRWRS-ABFSHIP Elverta Diversion Alternative) would run through a portion of the levee. The two 66-inch-diameter pipeline would each increase to 78-inch-diameter and continue east along Elverta Road to a new WTP to be constructed just north of Elverta Road. (The 72-inch-diameter pipeline from the Joint SRWRS-ABFSHIP Elverta Diversion Alternative would run through a portion of the levee and then discharge into the adjacent NMWC canal.)

From the WTP, a 96-inch-diameter pipeline would convey treated water east along Elverta Road, and parallel to the Natomas East Drainage Canal (NEDC), before following the NEDC 4.5 miles south, parallel to Natomas Boulevard/Truxel Road, to connect to the existing Sacramento distribution system at Del Paso Road.

In addition, a pipeline, initially 72 inches in diameter, would lead east from the WTP along Elverta Road, crossing under the NEDC, Steelhead Creek, and the Union Pacific Railroad (UPRR) to Sorento Road. The alignment would then turn north along Sorento Road, which turns into Pleasant Grove Road after leaving Sacramento County, and forms the border between Sutter County to the west, and Placer County to the east. After the pipeline reaches Riego/Baseline Road, it would turn eastward along Baseline Road before turning north along Fiddyment Road.

The pipeline would then extend north along Fiddyment Road, crossing under Pleasant Grove Creek and connecting to the existing PCWA distribution system at Athens Road.

A 30-inch-diameter pipeline would branch off the pipeline at the intersection of Baseline Road and Old Walerga Road, and run south, crossing Dry Creek, and continuing to Antelope Road where it would connect to the existing SSWD distribution system.

2.2. GEOTECHNICAL CHARACTERIZATION

In this section, sources for the geotechnical information in this chapter are listed, and regional geology and seismicity, hydrogeology and groundwater, and geotechnical conditions are described.

2.2.1. Sources of Geotechnical Information

Information for preparing this chapter was obtained by reviewing geotechnical reports prepared by others in conjunction with the following projects in the general vicinity of the components of each alternative:

- Lower Northwest Interceptor project (Sacramento Regional County Sanitation District (SRCSD))
- Upper Northwest Interceptor project (SRCSD)
- Titan 1-A Missile Facility (United States Army Corps of Engineers (USACE))
- American River Pump Station Project (PCWA)

In addition, maps and reports published by the California Geological Survey (CGS), and the United States Geological Survey (USGS) were reviewed. These and other sources used to prepare this chapter are listed below:

- Bartow, J.A., and E.J. Helley. 1979. Preliminary Geologic Map of Cenozoic Deposits of the Folsom Area, California. USGS.
- Carlson, W. 1990. Auburn Dam Interim Construction Geology Report, Auburn, California. For Reclamation. November.
- Duffield, W.A., and R.V. Sharp. 1975. Geology of the Sierra Foothills Melange and Adjacent Areas, Amador County, California. USGS.
- Espana Geotechnical Consultants. 2001. Preliminary Geotechnical Report for the Lower Northwest Interceptor Project, Sacramento and Yolo County, California. For MWH. September.
- Espana Geotechnical Consultants. 2002. Final Geotechnical Report for the Upper Northwest Interceptor, Section 7 – Sacramento County/City of Citrus Heights, California. For HDR. October.
- Helley, E.J., and D.S. Harwood. 1985. Geologic Map of the Late Conozoic Deposits of the Sacramento Valley and Northern Sierra Foothills, California. USGS.

- Jennings, C.W. 1994. Fault Activity Map of California. USGS.
- Kleinfelder. 2003. Geotechnical Data Report – New Natomas Pump Station, Lower Northwest Interceptor Project, Sacramento, California. For HDR.
- Kleinfelder. 2003. Geotechnical Data Report – Natomas Force Main, Lower Northwest Interceptor Project, Sacramento, California. For Black and Veatch.
- Kleinfelder. 2003. Geotechnical Data Report – Northern Sacramento River Crossing, Lower Northwest Interceptor Project, Sacramento, California. For Hatch Mott MacDonald.
- Mark Group. 1998. Draft Report Phase 2 Geotechnical Services for Final Design, American River Pump Station Project, Placer County Water Authority, Auburn, California.
- MWH. 2000. Lower Northwest Interceptor Design Report, Sacramento and Yolo Counties, California. For Sacramento Regional County Sanitation District (SRCSD). September.
- Wagner, D.L. 1981. Geologic Map of the Sacramento Quadrangle, California. USGS.
- Woodward Clyde. 1997. Focused Remedial Investigation Work Plan Titan 1-A Missile Facility, Lincoln, California. For USACE. May.

Field investigations to support preparation of this chapter consisted of a surficial reconnaissance of the proposed location of the facilities associated with each alternative.

2.2.2. Regional Geology

The study area is situated in the eastern portion of the Sacramento Valley, which includes the northern portion of the Great Valley Geomorphological Province of California. **Figure 2-1** is a surficial geologic map of the region with the features of the alternatives superimposed for reference.

The Great Valley of California is approximately 400 miles long and 40 miles wide, oriented along the axis of the State. Erosion of the Coast Ranges to the west and Sierra Nevada mountains to the east has generated alluvial, overbank, and localized lacustrine sediments, which have been deposited in the valley to a thickness of as much as 50,000 feet. Subsequent deformation folded these sediments into an asymmetric syncline with its axis off center toward the Coast Ranges. Along the eastern boundary of the Sacramento Valley, these alluvial deposits pinch out where they lap onto older alluvial deposits associated with western-flowing streams emanating from the foothills of the Sierra Nevada.

The portion of the project area within the Great Valley Geomorphological Province has been mapped in great detail, most recently by Helley and Harwood (1985). Map units include Holocene sediments characterized by active river channel deposits (Q) along the Sacramento and American rivers, alluvium (Qa) representing pre-levee and overbank deposits along the former meandering natural channels of the Sacramento River, and basin deposits (Qb) characterized by floodplain sediments outside the former Sacramento River channels. These deposits overlay relatively older Pleistocene deposits such as the Modesto (Qm) and Riverbank (Qr) formations, which pinch out to the east against the Turlock Lake Formation (Qtl), which consists of alluvial fan material associated with western-flowing rivers and streams from the Sierra Nevada. **Table 2-1** describes the stratigraphy of this portion of the study area.

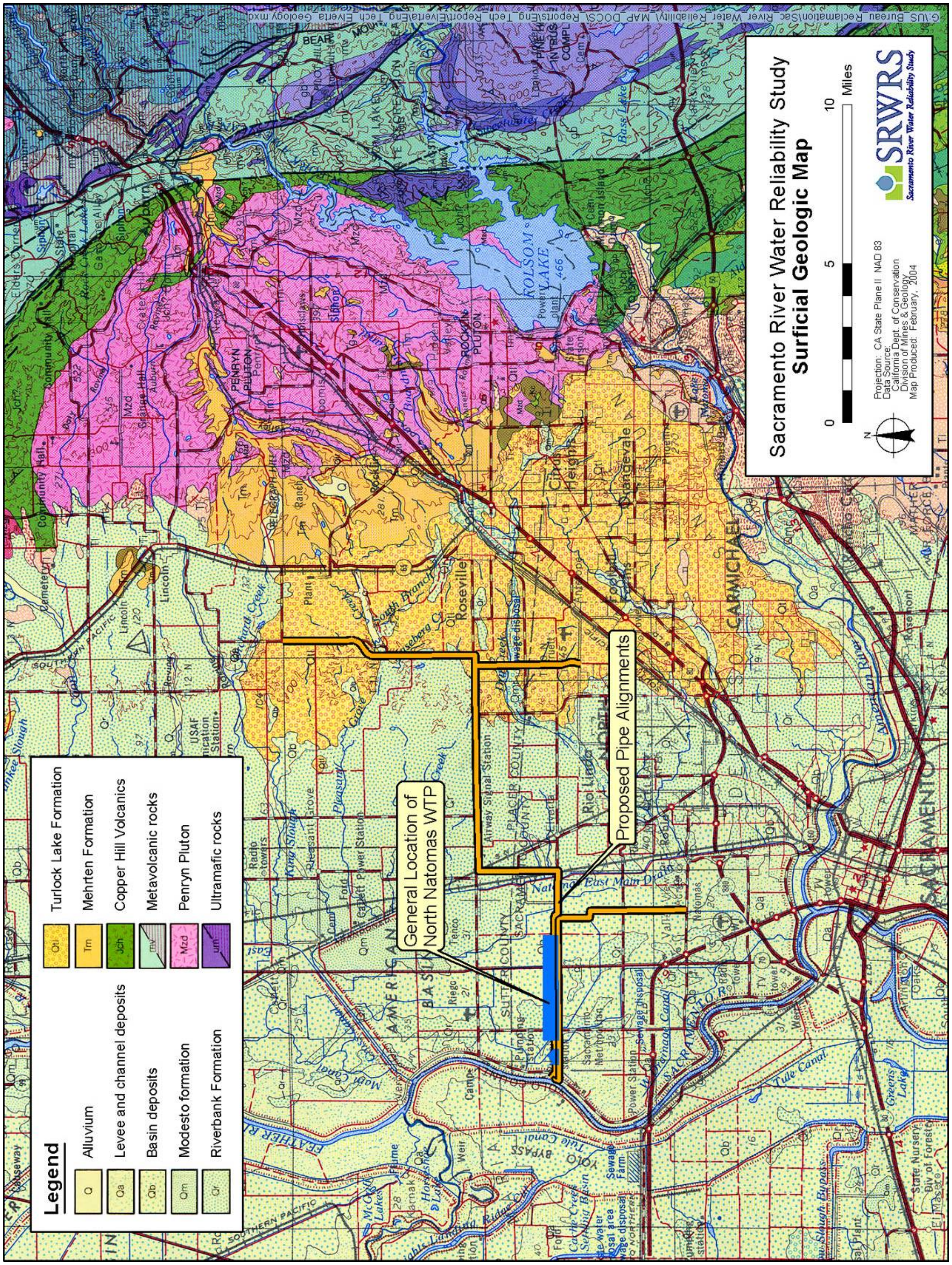


Figure 2-1 Surficial Geologic Map

Table 2-1 Stratigraphy of the Northeastern Portion of the Great Valley Geomorphological Province

AGE	FORMATION	MAP SYMBOL(1)	DESCRIPTION
Quaternary	Recent Alluvium and Levee Deposits	Q	Loose silty sand (SM), and well to poorly graded sand and gravel (SW-SP-GP) deposits in the current Sacramento River channel and against the levees.
	Basin Deposits	Qb	Layer 10 to 20 feet thick of dark, often organic, stiff to very stiff, silts (ML) and clays (CL). The basal contact of this unit is relatively uniform with the exception of apparent paleo-channels infilled with less clayey and more silty and sandy deposits. These alluvial sediments represent overbank floodplain deposits.
	Channel Deposits	Qa	Deposits 5 to 25 feet thick of loose to dense, silty sand (SM) and well to poorly graded sand (SW-SP) with localized layers or lenses of silt (ML). These sediments represent meandering channel deposits of the Sacramento and American rivers prior to levee construction.
	Modesto	Qm	From 0 (where they pinch out to the east) to as much as 60 feet thick of dense, well to poorly graded sands and gravels (SW-SP-GP) differentiated from overlying deposits primarily on the basis of density and gravel content (i.e., Qm denser and more gravelly than Qa). Absence of Qm to the east represents pinching out against alluvial fan deposits to the east.
	Riverbank	Qr	Stiff to dense silts (ML) and clays (CL) with minor lenses of dense poorly graded sands and gravels (SP-GP). Qr outcrops east of the Sacramento River and generally underlies Qb, Qa, and Qm sediments, and is thought to represent alluvial fan deposits transported by rivers emanating from the Sierra Nevada foothills.
	Turlock Lake	Qtl	Dense, relatively hard, partially consolidated silt (ML), poorly graded sand, and gravel (SP-GP) fan material derived mainly from Sierran granitic and metamorphic rocks.

Notes:
(1) Refer to Figure 2-1.

2.2.3. Regional Seismicity

Tectonically, the study area is relatively distant from major Holocene (last 10,000 years) active fault systems, as can be seen in the map of faults and historic earthquakes (**Figure 2-2**). Historic earthquake epicenters to the west of the project include the San Andreas and Hayward-Rodgers Creek fault systems, and the Coast Ranges-Sierran Block boundary system. To the east lie the Foothills Fault system and Sierra Nevada Frontal Fault system. Since the nearest active fault systems are a considerable distance from the site, recorded ground motions have been historically low. **Figure 2-3** is a map of peak ground acceleration (PGA) contours for the region. **Table 2-2** shows the major fault systems in the region, approximate distance from the center of the study area, and magnitude of a potential earthquake in the system.

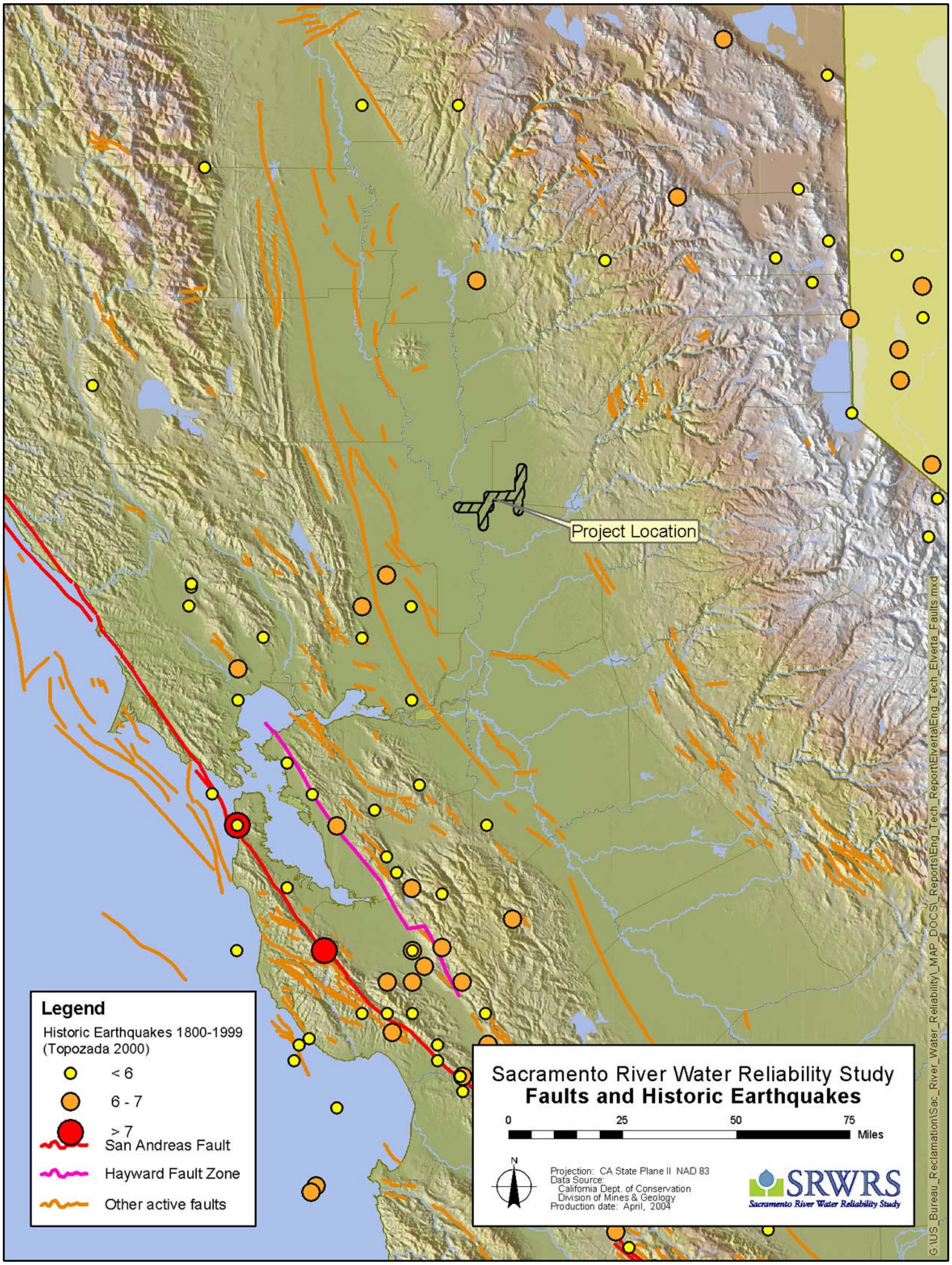


Figure 2-2 Faults and Historic Earthquakes

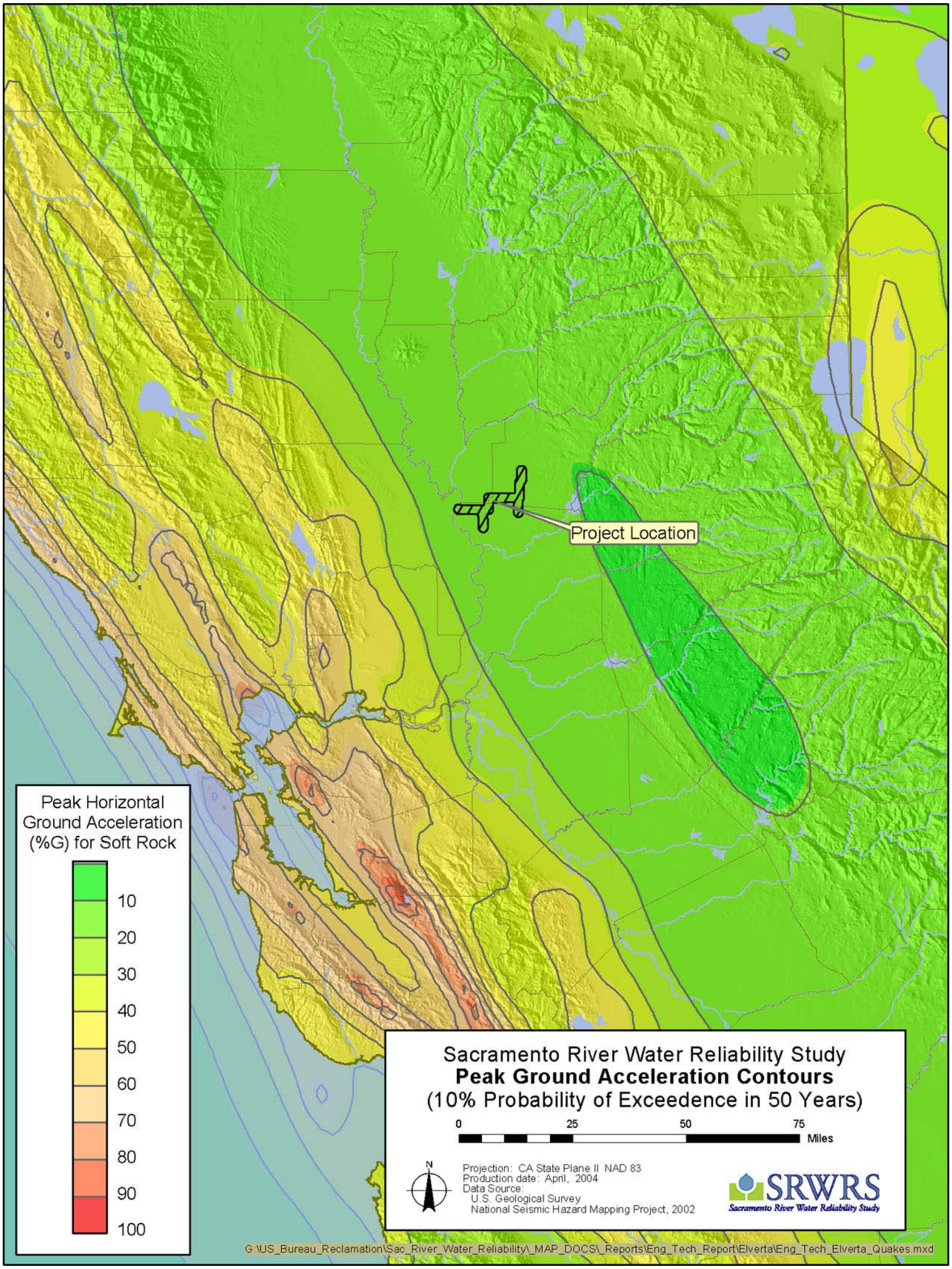


Figure 2-3 Peak Ground Acceleration Contours

Table 2-2 Regional Fault Systems

FAULT SYSTEM	DISTANCE (miles)	MAGNITUDE
Foothills Fault System	15	6.5
Dunnigan Hills	25	6.5
Coast Range – Sierran Block Boundary	25	6.8
Hayward – Rodgers Creek	70	7.1
San Andreas	90	8.0

The potentially active faults nearest to the study area are associated with the Foothills Fault system immediately to the east within the Foothills Melange-Ophiolite Metamorphic Belt. This series of subparallel, northwest-trending vertical faults includes at least two major fault zones. The easternmost is the Melones Fault zone, and the westernmost is the Bear Mountains Fault zone. The Foothills Fault system is approximately 200 million years old, with the last major seismic movement occurring about 140 million years ago. Although the Willows and Dunnigan Hills faults have been mapped a relatively short distance to the west of the study area, these faults are not classified as active by CGS, and are thus not considered capable sources of potential earthquakes or ground rupture.

2.2.4. Hydrogeology and Groundwater

As described above, the surficial geology of the western portion of the study area comprises recent alluvial deposits adjacent to the Sacramento River. In general, the hydrogeologic condition of these deposits is characterized by a nearly continuous surface layer 10 to 20 feet thick of low-permeability, soft to stiff clays and silts, underlain by a layer 5 to 25 feet thick of slightly dense to dense sand conducive to relatively high storage and flow of groundwater. Beneath these two layers lie the considerably older, denser, and less permeable sand, gravel, and stiffer silts and clay of the Modesto, Riverbank, and Turlock Lake formations.

Groundwater levels in the western portion of the study area are primarily controlled by natural recharge from the American and Sacramento rivers to the south and west, respectively, and the Natomas Drainage Canal system near the central and eastern portion of the area. Discharge of the aquifer has historically occurred as a result of agricultural irrigation using groundwater pumping wells.

Groundwater levels in the area generally range from about 2 to 5 feet above mean sea level (msl), or about 7 to 15 feet below the ground surface. However, historic records of the California Department of Water Resources (DWR) from 1963 to 2003 indicate groundwater levels may be as high as the ground surface concurrent with high Sacramento River and American River levels during major storm/flood events, such as in 1986 and 1997. Groundwater levels are expected to vary based on seasonal influences, adjacent canal or river stage, irrigation practices, runoff conditions, and other factors. Groundwater contours drawn from spring 2002 data are shown in **Figure 2-4**.

Although indications of groundwater contamination have not been encountered in the study area, pesticide, oil/petroleum hydrocarbon products, and methyl tertiary butyl ether (MTBE) are known to have affected groundwater in the vicinity of the Sacramento International Airport, truck stops along major highways, and Titan missile silos in Lincoln.

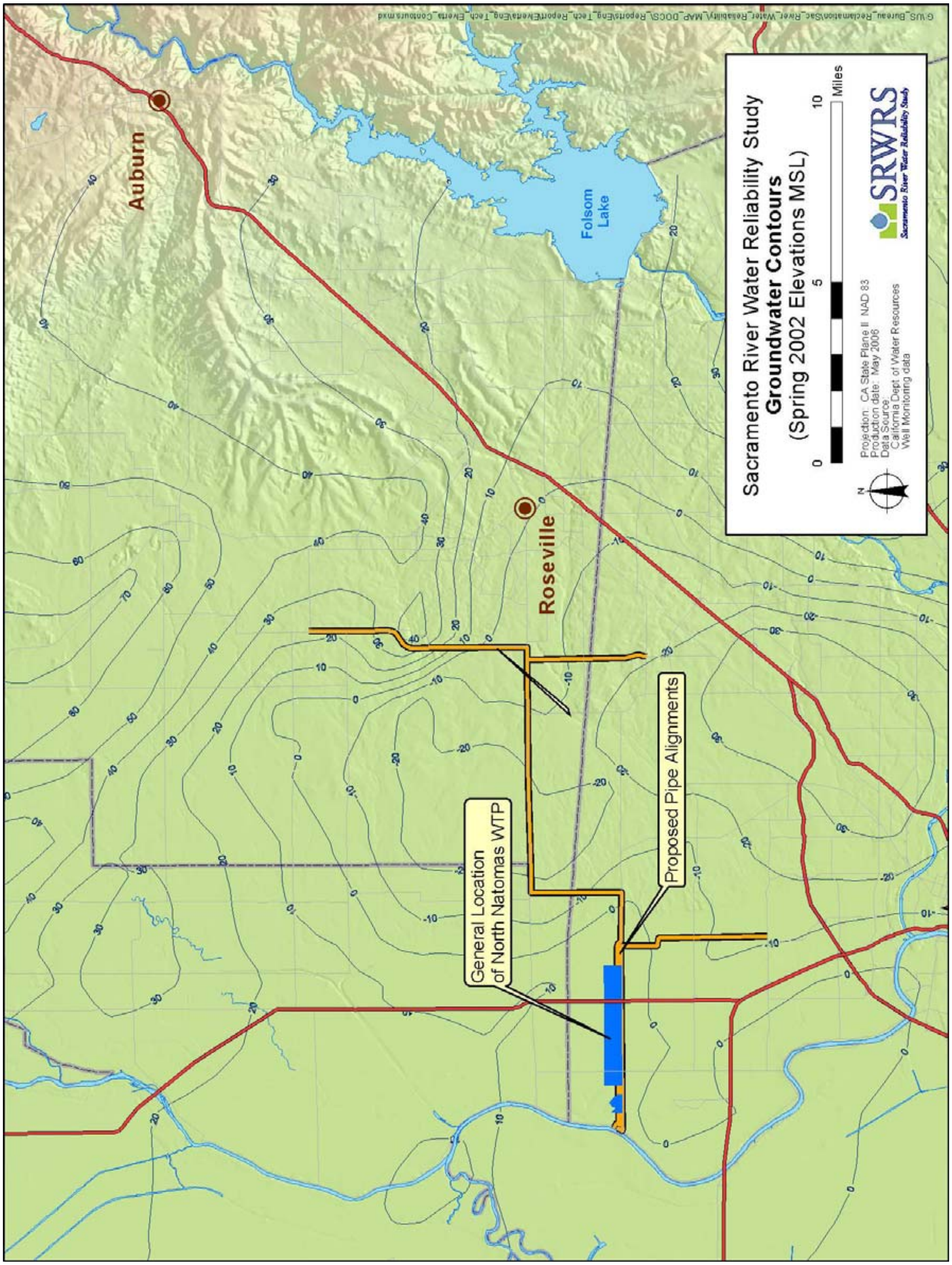


Figure 2-4 Groundwater Contours

2.2.5. Geotechnical Conditions

The Elverta Intake structure would be located in recent Quaternary alluvium of the Sacramento River channel, which consists of sand, silty sand, and gravel. The raw water pipeline would pass through the levee composed of sand and silty sand, and then along the low-lying fields adjacent to Elverta Road. These fields are primarily basin deposits characterized by a layer of clayey, relatively impervious soils approximately 10 to 20 feet thick overlying more pervious alluvial sands.

Depending on where it is located along Elverta Road, the North Natomas WTP site could be founded in relatively soft clay, silty sand, and sandy basin deposits or firmer deposits of the same materials associated with the Riverbank Formation.

The 96-inch and 72-inch-diameter pipelines would continue in parallel east in basin deposits or the Riverbank Formation. The 96-inch pipeline that turns south to connect to Sacramento's distribution system would be entirely within basin deposits. The 72-inch-diameter pipeline would continue east, crossing under Steelhead Creek, which is located approximately at the contact between basin deposits to the west and the Riverbank Formation to the east. The pipeline would then turn north, just east of the canal, to Riego/Baseline Road, where it would turn east, and then north again on Fiddyment Road, all within the Riverbank Formation. North of Pleasant Grove Creek, the pipeline is expected to be in the partially consolidated sand, silt, and gravels of the Turlock Lake Formation until the PCWA connection at Athens Road. The pipeline south to the SSWD connection would run approximately along the north-south contact between the Riverbank Formation to the west and the Turlock Lake Formation to the east until turning east entirely within the Turlock Lake Formation.

2.3. CONSTRUCTION CONSIDERATIONS

Groundwater is expected to be the main construction consideration for the pipelines in the flat western part of the study area adjacent to the Sacramento River. Excavations for the pipelines and North Natomas WTP would have to be dewatered where groundwater level was above the pipe trench or structure invert and the deep excavations shored. Where trench invert is projected to be within the upper impervious zone, care would have to be taken to ensure that enough material is left in the bottom of the trench excavation to offset uplift pressure from the underlying confined aquifer. This situation is expected to continue until about Pleasant Grove/Sorento Road. From that point east, trench excavation should be generally in the dry.

Crossing of Highway 99 and the UPRR tracks likely would be by double-pass tunneling methods in which 84-inch- and/or 120-inch-diameter steel casings would be jacked behind excavation by a tunnel-boring machine (TBM). The 72-inch and/or 96-inch carrier pipes would be grouted inside the casings. The 72-inch pipeline crossing of the NEDC is expected to be installed by single-pass tunneling methods.

2.4. GEOTECHNICAL HAZARDS

Geotechnical hazards discussed in this section include earthquakes, groundwater, slope stability, hazardous gases, and potentially corrosive soils.

2.4.1. Earthquakes

Aspects of earthquakes to be considered include seismic ground motions, surface rupture, and liquifaction.

2.4.1.1. Seismic Ground Motions

Ground motions are estimated by modeling the behavior of the source fault(s), the travel path to the site, and near-surface conditions beneath the site. This can be accomplished either by assuming an earthquake occurs at each source fault (i.e., deterministically) or by estimating the likelihood and understanding of an event given the fault(s) movement and seismic history (i.e., probabilistically). Most California agencies and the current Uniform Building Codes (UBC) prefer the probabilistic method. This method has been used by CGS (1996) and USGS (1996) for the entire State for soft rock conditions. As shown in **Figure 2-3**, this Probabilistic Seismic Hazard Analysis (PSHA) modeling estimates a maximum horizontal PGA of 0.2g for the overall project area using the Design Basis Earthquake (DBE) (10 percent probability of exceedence in 50 years) ground motion.

2.4.1.2. Surface Rupture

The potential for ground surface rupture is generally assessed on the basis of the presence of active Holocene (less than 10,000 years) faulting in the project area. Since no active faults have been mapped in the study area, and the site is not located within or near an Alquist-Priolo Earthquake Fault Zone, surface rupture is not considered a hazard for any of the planned features.

2.4.1.3. Liquefaction

Liquefaction is a condition that occurs when relatively low-density, saturated soils behave as a fluid if subjected to seismic ground motions. This condition is most prevalent in loose, granular soils within 50 feet of the ground surface. The principal effects of liquefaction on buried pipelines or structures are settlement (both total and differential), loss of foundation support, buoyancy, and lateral spreading of soils near free faces such as levees. Since low-density granular soils are known to exist beneath the western portion of the study area in conjunction with high groundwater levels, liquefaction cannot be ruled out in this area, and a liquefaction analysis should be performed during the next phase of study.

2.4.2. Groundwater

Shallow groundwater conditions are common in the western portion of the study area, especially adjacent to the Sacramento River and NEDC. Seepage from the Sacramento River and NEDC through relatively permeable sandy materials overlain by less permeable clayey soils is expected to cause locally confined aquifer conditions during periods of elevated river levels. Aquifer confinement occurs when the piezometric groundwater surface elevation is above the bottom of a confining clay layer (aquitard). Near the Sacramento River east levee, the piezometric groundwater surface elevation is expected to be above the ground surface during periods of high river levels and decrease with distance from the river. These high groundwater conditions could result in unstable excavation bottoms and side slopes unless excavations are properly dewatered or stabilized by shoring.

2.4.3. Slope Stability

Due to the flat topography of the western portion of the study area, potential for landslides and/or lateral spreading during a seismic event would be confined to existing levee slopes of the Sacramento River and the NEDC. The stability of these slopes would depend on the height and steepness of the slope versus the strength of underlying materials, and should be analyzed in conjunction with proposed excavations exposing prelevee alluvium. The stability of levee slopes should be calculated considering both static stability (i.e., no seismic loading) and seismic stability considering the anticipated 0.2g PGA for DBE ground motion. The potential for rapid drawdown conditions in the waterways should be considered and addressed as appropriate. Slope stability evaluation should also consider the potential for lateral spreading toward free faces represented by the Sacramento River and the NEDC.

2.4.4. Hazardous Gases

Hazardous subsurface migration of gases such as methane has become a severe concern in some areas, especially adjacent to landfills, and oil and natural gas fields. Auburn Placer Disposal Service (APDS) operates a landfill in conjunction with the Western Placer Waste Management Authority's (WPWMA) Materials Recovery Facility (MRF) located south of Athens Avenue near the intersection with Fiddyment Road. Future geotechnical investigations of the pipeline alignment in this area should include a Phase I Environmental Site Assessment to detect the presence of hazardous gases. Gas fields do exist within the region, but none are mapped in the study area. No oil fields are located in the greater Sacramento region.

2.4.5. Potentially Corrosive Soils

Recent tests for soluble sulfates, soluble chlorides, and electrical resistivity of soils in the western part of the study area in support of SRCSD's Lower Northwest Interceptor project indicated these soils to be moderately corrosive to buried metal pipe. Mitigation measures would typically include bonding of pipe joints and construction of test stations along the pipeline alignments to monitor local corrosion conditions. Cathodic protection of portions of the pipeline may be required.

2.5. RECOMMENDED FUTURE GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations for the next phase of project development should include the following:

- Detailed surficial geologic mapping
- Preliminary subsurface investigation through boreholes and test pits of the Sacramento River intake structure, WTP site, and pipeline alignments
- Sample collection and laboratory testing
- Retention of a corrosion engineer to conduct a Soil Corrosivity Investigation and produce a report
- Comprehensive analysis of available groundwater data and seasonal fluctuation of groundwater levels
- Phase 1 environmental assessment of groundwater quality to identify any hazardous conditions that should be avoided, and to provide baseline information for dewatering permit applications
- Phase 1 Environmental Site Assessments

CHAPTER 3 INTAKE FACILITY AND FACILITIES REQUIRED FOR THE JOINT SRWRS-ABFSHIP ELVERTA DIVERSION ALTERNATIVE

This chapter presents an engineering analysis of the intake facility for the SRWRS Elverta Diversion Alternative as part of the SRWRS. This chapter is a continuation and refinement of the work presented in Appendix C of the SRWRS Phase I Report. The purpose of this chapter is to advance the engineering development of the intake facility and describe it to a sufficient extent to allow completion of the project BA and EIS/EIR. All elevations presented in this chapter are referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29).

The SRWRS Elverta Diversion Alternative developed in the Phase I Report (Appendix C to the Initial Alternatives Report, (March, 2005)) included construction of a single new intake facility located on the Sacramento River. The proposed facility is currently to provide 235 mgd capacity and supply all SRWRS cost-sharing partners in the following distribution: PCWA at 65 mgd, SSWD at 15 mgd, Roseville at 10 mgd, and Sacramento at 145 mgd. This chapter presents the basis of design, the site evaluation selection process, a refinement of the river hydrology, and the intake configuration evaluation process.

In addition to the base alternative described above, the Phase I Report briefly discussed the possibility of consolidating intake facilities with NMWC, which planned to expand its existing intake, located on the Sacramento River near the proposed Elverta Intake site, to 135 mgd (210 cfs). This chapter will refine the consolidation discussion and present this subalternative, known as the Joint SRWRS-ABFSHIP Elverta Diversion Alternative, which includes increased pumping capacity and canal improvements required for NMWC.

The intake site evaluation and selection activities described in this section were developed using the intake facility required for the SRWRS Elverta Diversion Alternative. Modifications and additional facilities required for the Joint SRWRS-ABFSHIP Elverta Diversion Alternative are presented in **Section 3.7**.

This chapter concludes with a discussion of the power, sewer, storm drainage, and special considerations at the proposed facilities. Construction and operating characteristics of the proposed facilities are also presented.

3.1. BASIS OF DESIGN

The initial criteria used as the basis of design for the intake facility are based on the SRWRS cost-sharing partners' operational requirements, current published criteria for fish passage facilities by CDFG (1997)[2] and the National Oceanic and Atmospheric Administration (NOAA), National Marine Fisheries Service (NOAA Fisheries, 1997),[3] current industry practice, and experience at similar facilities. Criteria are presented below. Criteria are presented below.

3.1.1. Project Flows and Pump Configuration

Criteria for project flows and pump configuration are shown for both the SRWRS Elverta Diversion Alternative and the Joint SRWRS-ABFSHIP Elverta Diversion Alternative.

[2] CDFG. 1997. Fish Screening Criteria. April.
[3] NOAA Fisheries. 1997. Fish Screening Criteria for Anadromous Salmonids. January.

3.1.1.1. SRWRS Elverta Diversion Alternative

- Maximum water diversion = 235 mgd (365 cfs).
- Minimum water diversion = 66 mgd (102 cfs).
- Pump configuration could include two @ 11 mgd, two @ 22 mgd, five @ 33 mgd; some or all of these may be equipped with variable-frequency drives. One additional 33-mgd pump will be provided for backup.

3.1.1.2. Joint SRWRS-ABFSHIP Elverta Diversion Alternative

- Maximum overall water diversion = 371 mgd (575 cfs).
- Additional four dedicated pumps @ 33 mgd each, with variable-frequency drives, provided for NMWC.

3.1.2. Fisheries and Fish Screens

- The project design will be based on protecting juvenile anadromous fish present in the Sacramento River at the point of diversion.
- The target species and its life stage of concern are assumed to be the winter-run Chinook salmon fry.
- River water approach velocity, normal to the screen face, will be 0.33 feet per second (fps) maximum.
- River sweeping velocities parallel to the screen face must be at least twice the approach velocity.
- The screen opening slot will be 1.75 millimeters (mm) wide (0.069 inches).
- Stainless steel wedgewire screens will be used.
- A screen cleaning mechanism designed to clean all screens within a 5-minute period will be used.

3.1.3. Debris Management

- Intake structure and intake access bridge will be designed to shed debris.
- Intake structure and intake access bridge will be designed to withstand high impacts from large floating or submerged debris.

3.1.4. Levee Impacts

- The levee will be restored in accordance with The Reclamation Board of the State of California (Reclamation Board) levee design standards.
- The levee road (Garden Highway) will be restored and/or modified in accordance with current Sacramento County Department of Transportation design standards.
- Consultation with The Reclamation Board will take place as part of the refinement of intake alternatives to verify design and construction constraints.

3.1.5. Operation and Maintenance

- Intake facility will be unmanned.
- The project will provide means for accessing and removing fish screens and pumps for maintenance and repair.

3.1.6. Water Supply Reliability

- Intake will be designed to provide the desired flows on a continuous basis throughout the year.
- The completed project will operate at varying water levels and flow in the Sacramento River, with the range spanning the historical average low flow through the 100-year-flood flow.

3.1.7. Environmental Impacts

- Intake facility design will strive to minimize impacts to the riparian zone, aquatic habitat, and the shaded river habitat.
- Design will strive to minimize facility footprint by maximizing use of available water depth.

3.1.8. Public Safety

- The facility will be designed to minimize impacts on river traffic and recreation during construction and operation.
- Facilities will be designed with consideration of published guidelines from the United States Coast Guard (USCG) and the California Department of Boating and Waterways.

3.1.9. Security

- Design of the intake facility will consider the security of the structure and its components relative to theft and vandalism.
- Design will assume a motorized, spiked, or barbed-wire-topped gate on the bridge and alarms on the gate and doors.

3.1.10. Regulatory Requirements

Planning and design will follow published guidelines for all pertinent governmental agencies, including, but not limited to, the following:

- Reclamation
- USACE
- CDFG
- USCG
- Federal Aviation and Administration (FAA)
- Sacramento Area Flood Control Agency (SAFCA)
- The Reclamation Board

3.2. SITE EVALUATION AND SELECTION

The intake site selection process presented in the Phase I Report included a 3-mile reach of river in the vicinity of the location of the proposed intake site on the Sacramento River at river mile (RM) 74.6, initially identified by Sacramento. Bathymetric and topographic information developed by USACE, preliminary river flow/stage analysis, aerial photography, and field investigations were used to evaluate the river reach. It was determined that the proposed site at RM 74.6 had the best design characteristics in the 3-mile reach evaluated.

To verify that other desirable sites on the river were not being overlooked, the current study evaluated a larger portion of the Sacramento River. An approximate 16-mile reach of river, from the confluence of the Sacramento and American rivers at approximately RM 60, to the Sacramento and Sutter county line at approximately at RM 76, was evaluated. This portion of river was selected because it represents the feasible boundaries of the project, based on proximity to the proposed service area and location relative to existing intakes (Sacramento's existing Sacramento River WTP Intake is located immediately downstream from the American River confluence).

3.2.1. Site Evaluation Criteria

Potential sites in the 16-mile reach were evaluated with respect to the criteria listed in **Table 3-1**.

Table 3-1 Intake Site Evaluation Criteria

Criteria	Evaluation Approach
Available water depth	A greater water depth to river bottom is an asset as it allows the intake structure to be constructed deeper, with a smaller overall footprint in the river.
Site located on an outside bend	Since water is moving faster on the outside bend, the chance for sediment deposit and build-up is lower.
Narrow river section between defined levees	Locating the intake between defined levees reduces the chance that the river will meander away from the intake.
Proximity to existing homes	Locating the intake farther away from homes, buildings, and parks was preferred; construction noise, operating noise, and maintenance activities may cause neighbors to oppose construction, or request operational restrictions.
Proximity to turnout points	Proximity to the cost-sharing partners' turnout points reduces overall project cost.
Site located on the left (east) bank of the river	An obvious criterion; this avoids the need to tunnel conveyance piping under the Sacramento River.

3.2.2. Site Evaluation Process

The following paragraphs describe the process used to evaluate potential intake sites within the stretch of the Sacramento River from the Sacramento and Sutter county line at RM 76 to the confluence of the Sacramento and American rivers at RM 60.

Six large figures of the Sacramento River USACE bathymetry data were overlaid on color aerial photographs and printed at a scale of 1 inch to 300 feet. Each figure included a section of the river approximately 3 to 4 miles in length. The figures were evaluated for potential sites using the criteria from **Table 3-1**. Several sites that appeared to meet the evaluation criteria were identified and field-evaluated. Advantages and disadvantages of each site were summarized.

Reduced-scale copies of the original figures used in the evaluation are included as **Figures 3-1** through **3-6**. The river segments presented in each figure are evaluated in the following sections, including advantages and disadvantages of the potential sites.

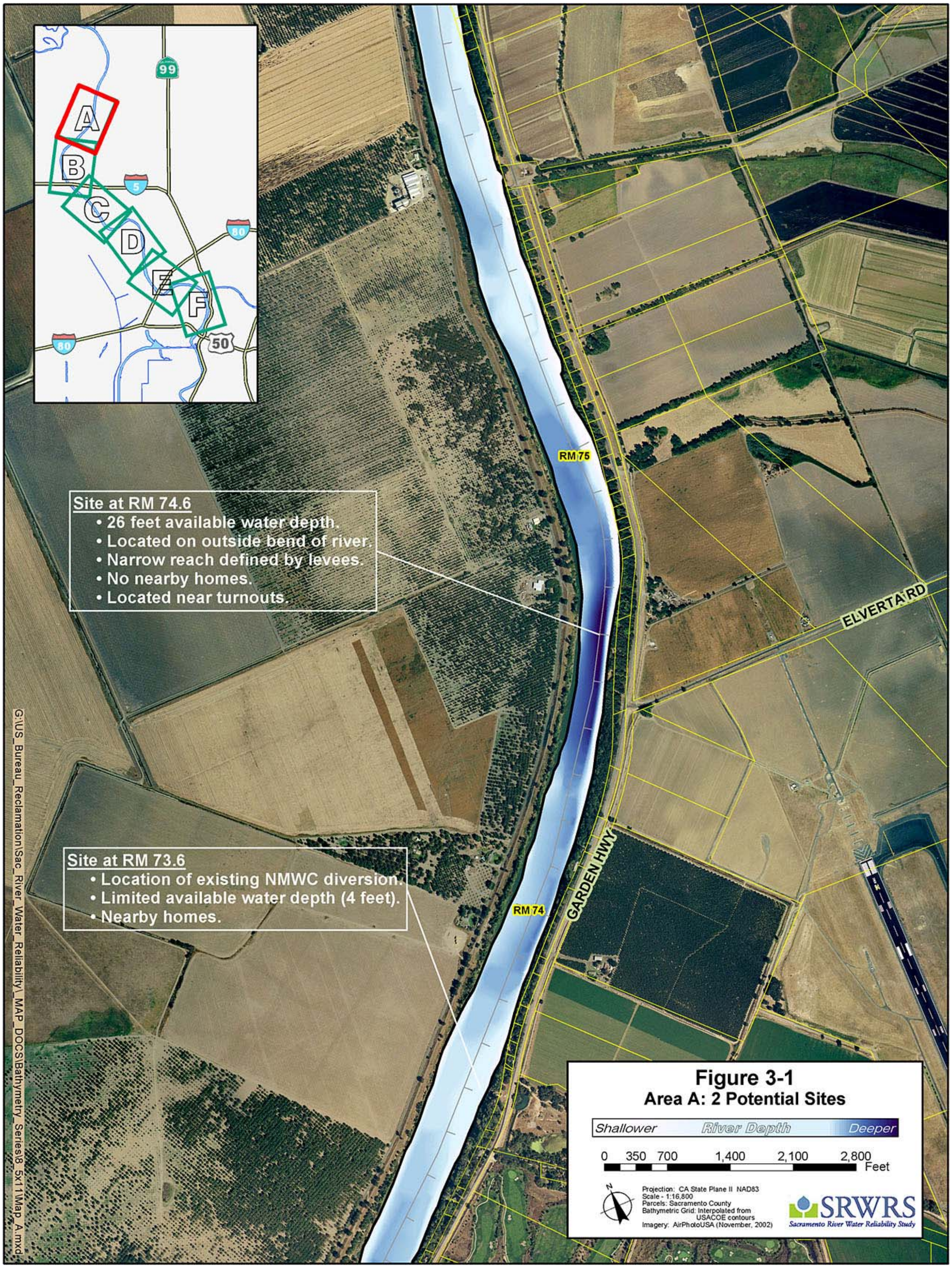


Figure 3-1 Area A: 2 Potential Sites

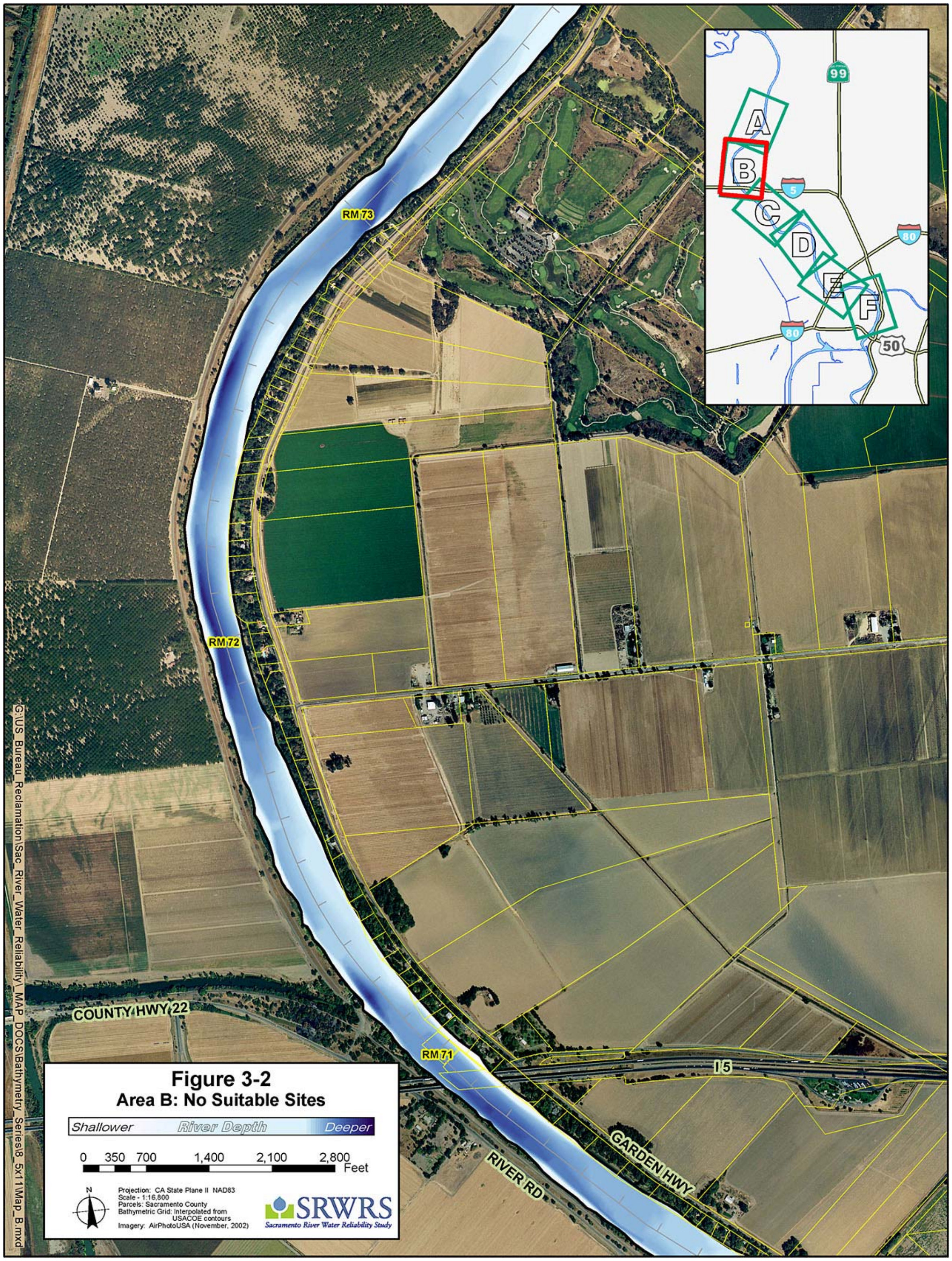


Figure 3-2 Area B: No Suitable Sites

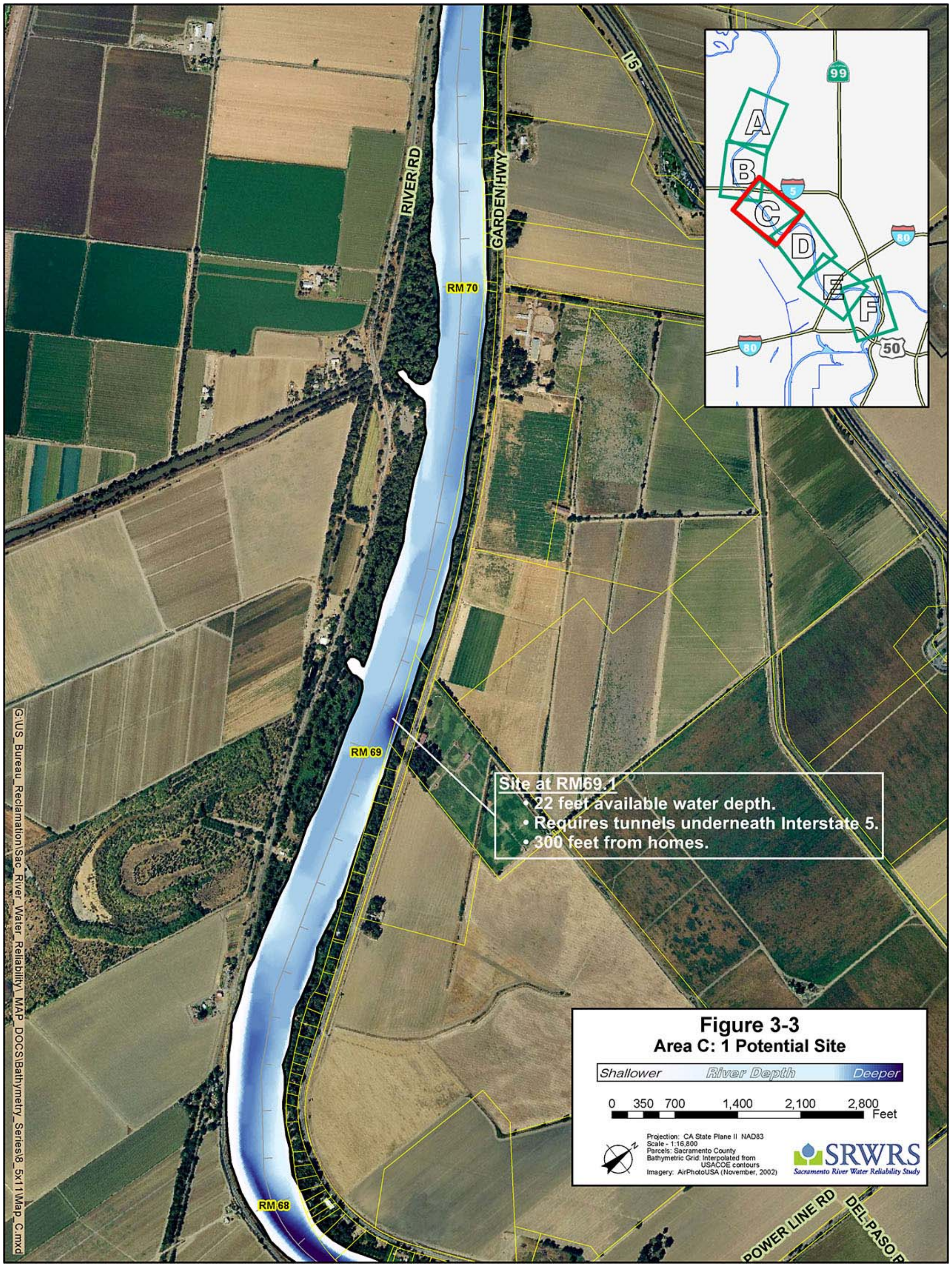


Figure 3-3 Area C: 1 Potential Site

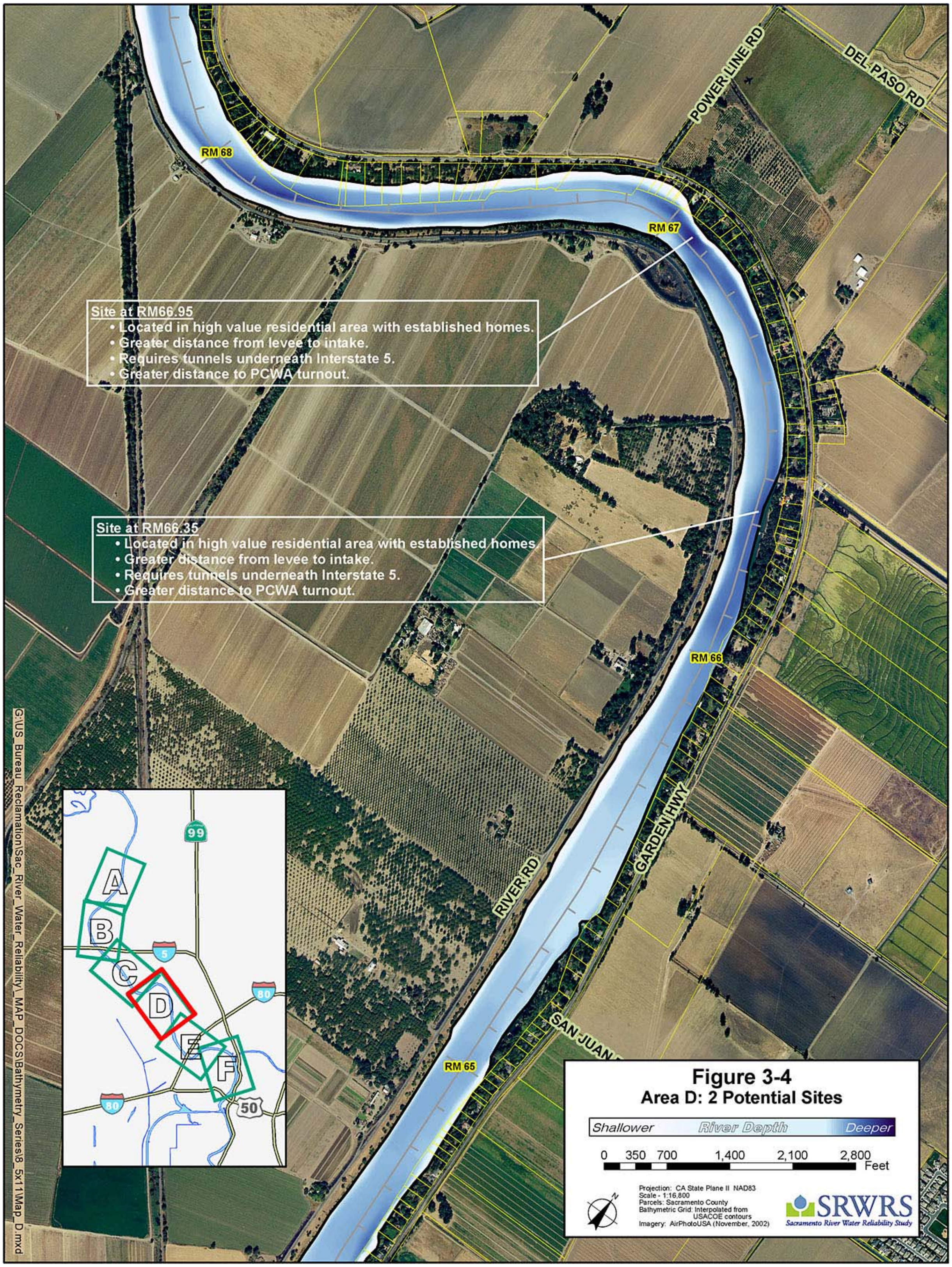


Figure 3-4 Area D: 2 Potential Sites

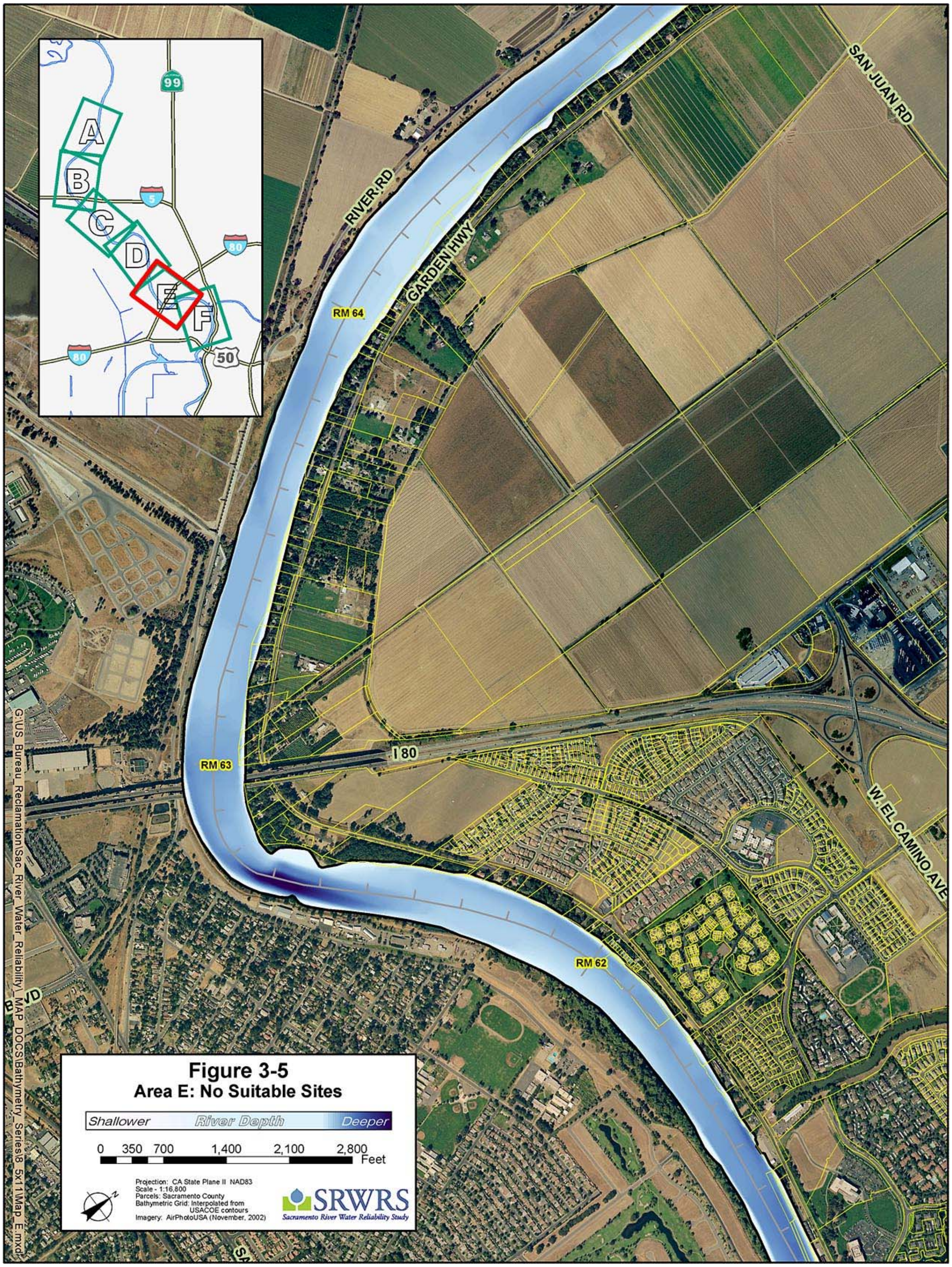


Figure 3-5 Area E: No Suitable Sites

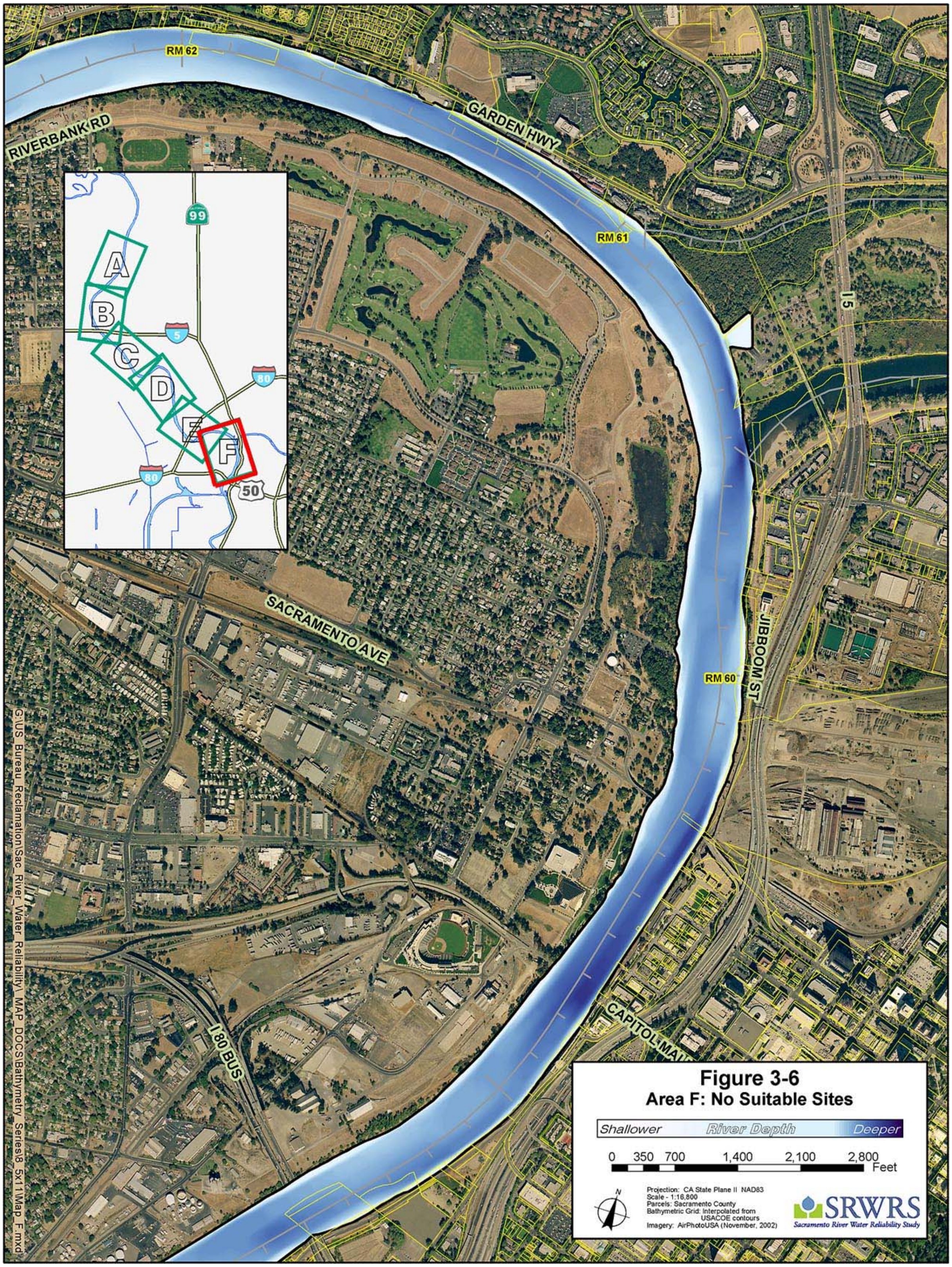


Figure 3-6 Area F: No Suitable Sites

3.2.2.1. RM 76 to RM 73.2

Two potential sites were identified in this reach. See **Figure 3-1**.

3.2.2.1.1. Site at RM 74.6

This is the site identified in the Phase I Report, initially referred to as the Elkhorn/Elverta Diversion Alternative intake site. A photograph of this site is shown in **Figure 3-7**. Advantages of this site include the following:

- Excellent available water depth of approximately 26 feet below low water level provides significant design flexibility and potential to reduce intake footprint.
- Located on an outside bend of the river with reduced risk of sediment buildup.
- Relatively narrow river segment between defined levees limits meandering.
- Located on land owned by Sacramento County near the high-noise Sacramento International Airport and away from existing homes.
- Proximal to two SRWRS cost-sharing partners' turnout points, approximately 10 miles to Sacramento's turnout at the intersection of Del Paso Road and Truxel Road and 22 miles to PCWA's turnout at the intersection of Athens Road and Fiddyment Road.

Figure 3-7 Potential Intake Site at RM 74.6

Disadvantages of this site include the following:

- Riparian habitat between the levee and the riverbank will be impacted by construction activities.
- Located within the Sacramento International Airport "Overflight Zone" and will require coordination with FAA.

3.2.2.1.2. Site at RM 73.6

This site is the existing NMWC intake near Elkhorn Boulevard and was initially considered in the Phase I Report because it was theorized that the cost-sharing partners could combine with NMWC to construct a new joint facility at this location. A photograph of this site is shown in **Figure 3-8**.

Figure 3-8 Existing NMWC Elkhorn Intake at RM 73.6

This site has one advantage:

- Located near cost-sharing partners' turnout points; distances similar to the site identified above at RM 74.6.

Disadvantages of this site include the following:

- Site has a limited available water depth of approximately 4 feet below low water elevation, which would increase the size and complexity of the intake structure.
- Residential homes are located to the north and south of the existing NMWC diversion at an approximate distance of 1,000 feet. The new structure would be substantially larger than the existing structure and affected neighbors could object to the project and/or demand engineering, architectural, or operational restrictions to the facilities.

3.2.2.2. RM 73.2 to RM 70.3

No suitable alternatives were identified in this portion of the river (see **Figure 3-2**). Disadvantages of this river segment include the following:

- Entire segment is an inside bend of the river, with associated low velocities and sediment deposition potential.
- Significant number of homes along the bank.
- Limited water depth available.

3.2.2.3. RM 70.3 to RM 67.8

One potential site was identified in this reach (see **Figure 3-3**).

3.2.2.3.1. Site at RM 69.1

A photograph of this site is shown in **Figure 3-9**. Advantages of the site include the following:

- Water depth available below low water elevation is approximately 22 feet.
- Located on an outside river bend, with associated low sediment build-up.
- Relatively narrow river segment between defined levees limits meandering.
- Deep water is available close to the levee (approximately 150 feet), which reduces impacts to riparian habitat.

Figure 3-9 Potential Intake Site at RM 69.1

Disadvantages of this site include the following:

- A small grouping of about four homes and the Christiana Farm (a horse breeding facility) are located directly across Garden Highway from the potential intake location (approximately 300 feet). Construction and operation of the intake facility would have a significant impact on this development. In addition, as will be discussed later in this chapter, the levee road would likely need to be raised 8 to 10 feet at the intake site to accommodate an access bridge. Raising the levee would cause it to extend farther landward, further encroaching on the existing development.
- Tunnel crossing of Interstate 5 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- While the distance to Sacramento's turnout at the intersection of Del Paso Road and Truxel Road decreases to 6 miles, the distance to PCWA's turnout at the intersection of Athens Road and Fiddyment Road increases to 28 miles.

3.2.2.4. RM 67.8 to RM 64.7

Two potential locations were identified in this reach (see **Figure 3-4**).

3.2.2.4.1. <u>Site at RM 66.95</u>

A photograph of this site is shown in **Figure 3-10**. Advantages of the site include the following:

- Water depth available below low water elevation is approximately 29 feet.
- Located on an outside river bend, with associated low sediment build-up.
- Relatively narrow river segment between defined levees limits meandering.

Figure 3-10 Potential Intake Site at RM 66.95

Disadvantages of this site include the following:

- Located in area of high-value private property. The majority of the homes in the vicinity of the site are 1- to 2-acre parcels containing large riverfront homes.
- Relatively long distance from the levee to the intake (approximately 450 feet) increases environmental and private property impacts.
- Tunnel crossing of Interstate 5 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- While the distance to Sacramento's turnout at the intersection of Del Paso Road and Truxel Road decreases to 5 miles, the distance to PCWA's turnout at the intersection of Athens Road and Fiddyment Road increases to 27 miles.

3.2.2.4.2. Sites in the Vicinity of RM 66.35

This section of the river, approximately 1,500 feet in length, could be used for an intake facility. Photographs of this site are shown in **Figure 3-11**. Advantages of the area include the following:

- Water depth available below low water elevation is approximately 21 feet.
- Located on an outside river bend, with associated low sediment build-up.
- Relatively narrow river segment between defined levees limits meandering.


Figure 3-11 Potential Intake Sites in the Vicinity of RM 66.35

Disadvantages of this area include the following:

- Located in area of high-value private property. The majority of the homes in the vicinity of the site are 1- to 2-acre parcels containing large riverfront homes.
- Relatively long distance from the levee to the intake (approximately 450 feet) increases environmental and private property impacts.

- Tunnel crossing of Interstate 5 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- While the distance to Sacramento's turnout at the intersection of Del Paso Road and Truxel Road decreases to 6 miles, the distance to PCWA's turnout at the intersection of Athens Road and Fiddyment Road increases to 28 miles.

3.2.2.5. RM 64.7 to RM 61.5

No suitable alternatives were identified in this portion of the river (see **Figure 3-5**). Disadvantages of this river segment include the following:

- Majority of the segment is an inside bend of the river, with associated low velocities and sediment deposition potential.
- Significant number of homes along the bank.
- Tunnel crossing of Interstate 5 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- Tunnel crossing of Interstate 80 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- Suboptimal available water depth.
- Suboptimal distance from turn-outs.
- Challenging routing of large-diameter pipelines through highly developed areas.

3.2.2.6. RM 61.5 to RM 60

No suitable alternatives were identified in this portion of the river (see **Figure 3-6**). Disadvantages of this river segment include the following:

- Significant number of homes and businesses along the bank.
- Tunnel crossing of Interstate 5 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- Tunnel crossing of Interstate 80 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- Proximity to Sacramento's existing Sacramento River WTP Intake might cause concern with regulators.
- Suboptimal distance from turn-outs.
- Challenging routing of large-diameter pipelines through highly developed area.

3.2.3. Conclusions and Site Selection

Five potential intake sites were identified on the Sacramento River between the Sutter County line and the confluence of the American River. Based on a review of the advantages and disadvantages of the sites presented above, it is clear that the site identified in the Phase I Report, located at RM 74.6, best meets the evaluation criteria presented in **Section 3.2.1**. A key advantage of this site is its location on land owned by Sacramento County that has limited uses due to its proximity to the Sacramento International Airport. The site can be developed with a manageable amount of environmental mitigation of riparian habitat. In addition, and importantly, the central location of the site with respect to the cost-sharing partners would help minimize project costs.

3.3. REFINEMENT OF RIVER HYDROLOGY

This section describes work done to determine appropriate Sacramento River water surface elevations to be used for design of the fish-screened intake at the preferred site at RM 74.6. The water surface elevation at the proposed site at any given time results from the interplay of a number of factors, including operation of the Fremont Weir and backwater effects of the American River. The methodology described below has been used successfully for design of intake structures on the Sacramento, American, and San Joaquin rivers. Additionally, initial assessments of design water surface elevations described herein should be further refined in the predesign phase of the project.

The design low water surface will define the elevation for the top of the fish screen, ensuring that it will be fully submerged and thereby ensuring also that mandated screen approach velocities will not be exceeded at the design flow rate. The 100-year flood elevation will be used to define the elevation above which in-river pumps and electrical must be located, and to define the elevation above which the underside of any access bridge must be located (with a minimum of 3 feet clearance). Intermediate water surface elevations will be developed for later use in detailed pump operation analysis. The following describes hydrologic data and modeling techniques used to determine the design water surface elevations.

River stage and flow records from USGS for the Sacramento River at Verona (Station No. 11425500, RM 78.3) were used to determine design flows for the project. An exceedence curve for Verona is presented in **Figure 3-12**. Hourly gage elevations recorded from January 1, 1990, to December 31, 2003, at Verona were averaged to obtain daily average stages. From the daily stage values, corresponding river flows were determined using Reclamation-provided rating curves. Although a larger period of record was available for flows at Verona, only the period after 1990 was used due to river system operational changes instituted for fish protection at this time as a result of the CVPIA.

The design low water surface elevation at the proposed site was determined by first ordering the daily stage data, in descending order, for the Verona gage station. It was noted that an elevation of 5.8 feet above msl (elevation 5.8) was recorded on a number of days and elevation 5.7 was recorded on several days. In addition, an elevation less than 5.7 was recorded only once in the 13-year period of record. Elevation 5.7, and its associated flow of 4,800 cfs, was selected as the low water design point.

To determine the corresponding elevation at the project site for the design low water flow determined above, a HEC-RAS computer backwater model was used. The model facilitated development of a rating curve, shown in **Figure 3-13**, for the river at the proposed Elverta Intake site. The design low water flow was then evaluated relative to the rating curve and a design low water elevation of 4.3 was established for the proposed site.

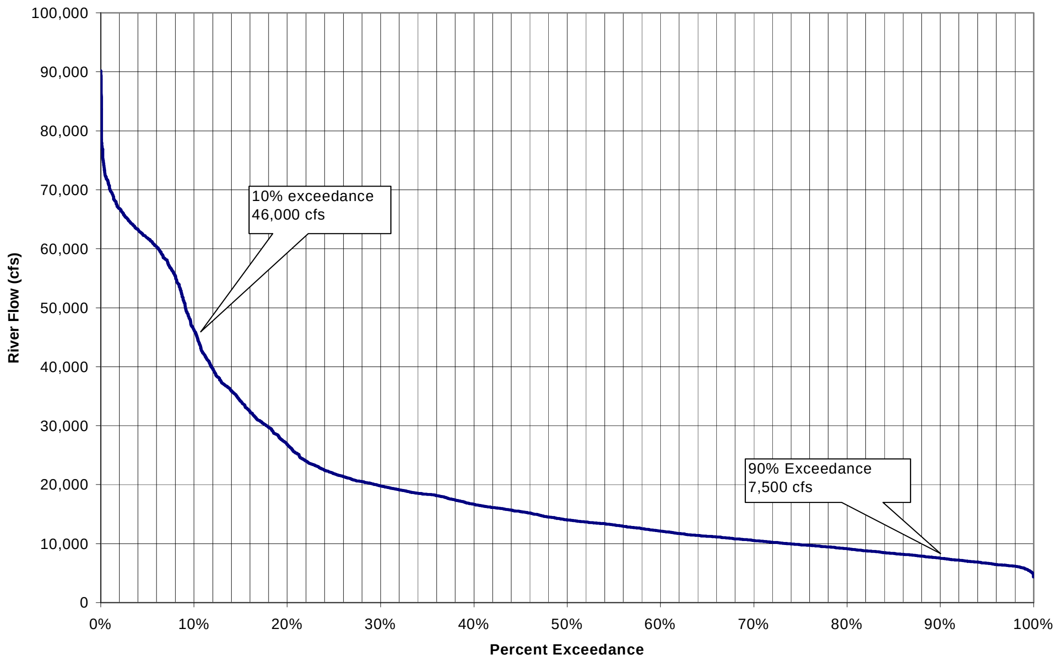


Figure 3-12 Exceedence Flow Curve for Verona (Station No. 11425500, RM 78.3)

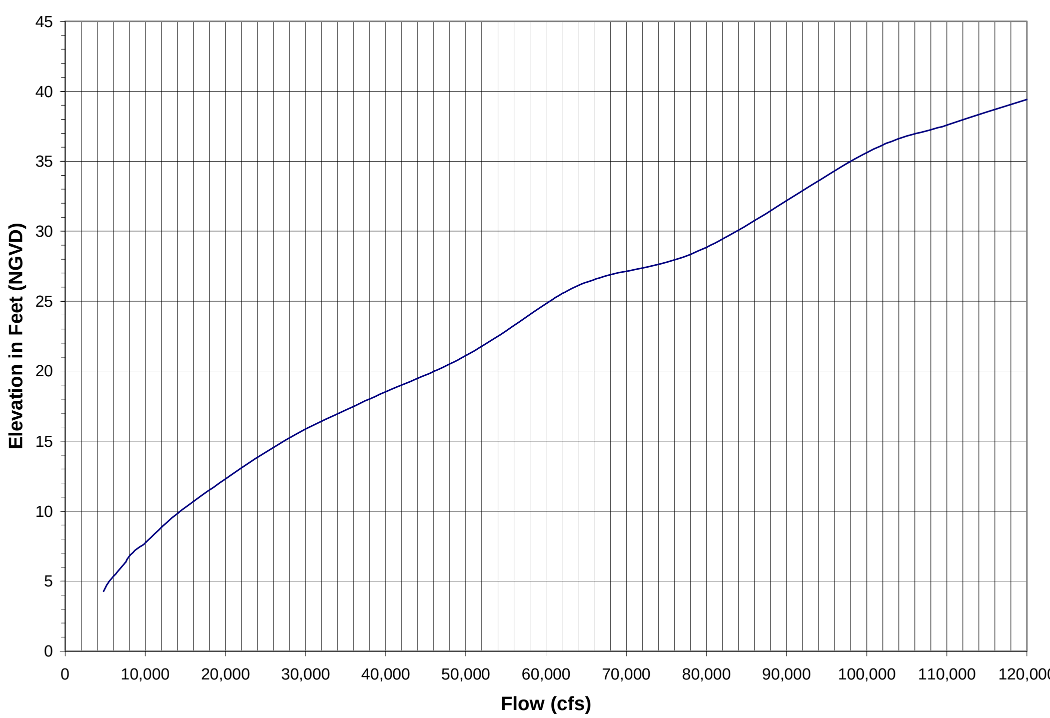


Figure 3-13 Rating Curve for the Proposed Elverta Intake Site – RM 74.6

Downstream boundary conditions used by the model were established using Sacramento River data measured at I Street (Station No. 11447500, RM 59.7). River cross section data used as the basis for the model geometry were taken from the USACE Sacramento and San Joaquin River Basins Comprehensive Study.[4] The geometry and the friction factor (Manning's n) were calibrated to match historical water surface elevations at the I Street and Verona gages.

The 100-year flood flow and stage, which is not impacted by CVPIA operational changes, was determined using river data for the period between 1967 and 2002. The USACE HEC-FFA (flood frequency analysis) model, based on the Guidelines for Determining Flood Flow Frequency, Bulletin No. 17B of the Hydrological Subcommittee, was used to calculate the flow. The analysis returned a 100-year flood flow of 120,000 cfs, which translated to a water surface elevation of 39.4 at the proposed Elverta site.

3.4. INTAKE CONFIGURATION EVALUATION AND SELECTION

This section presents the methodology and conclusions of the screening process for intake facility configuration alternatives for the proposed SRWRS Elverta Diversion Alternative intake site on the Sacramento River, as identified in the previous section.

3.4.1. Initial Development and Screening of Alternatives

As a first step in developing an intake configuration appropriate for the proposed project site, a design workshop was conducted and attended by MWH's leading fish-screened intake engineers. The workshop took place at the MWH offices in Bellevue, Washington, on November 17, 2003. Attendees included Dennis Dorratcague, Frank Postlewaite, and Clint Smith of the MWH Bellevue office, and Phil Salzman and M. Alejandro Salazar of the MWH Sacramento office. Workshop participants previously have been involved in design and construction of over 25 fish-screened intake facilities, ranging in size from 2 mgd to 1,600 mgd, and located in California, Washington, and Oregon.

The workshop included developing intake configuration alternatives, intake evaluation criteria, and a weighting system for the criteria, rating each alternative for its ability to meet each criterion, and scoring the alternatives based on the product of their ratings and the weightings. The methodology is summarized below.

3.4.1.1. Development of Intake Configurations Alternatives

The initial step of the evaluation process was to develop fish-screened intake structure alternatives applicable to the proposed Elverta site. Eleven conceptual alternatives were developed and are briefly described:

3.4.1.1.1. <u>Alternative 1 – Pier Intake Structure and Pump Station with Vertical Fish Screens on Two Sides</u>

This alternative would incorporate an oblong-shaped in-river intake structure and pump station oriented parallel to the river, located approximately 70 feet from the river bank at average flows (see **Figure 3-14**). The structure would rise to a height of approximately 60 feet above the water surface at average flow, about 30 feet higher than the top of the levee. The pump motors and electrical equipment would be located on a deck at an elevation safely above the 100-year flood elevation. Vertical flat panels of

[4] USACE. 2002. Sacramento and San Joaquin River Basins Comprehensive Study, Technical Studies Documentation. December.

stainless steel wedgewire fish screens would be located at the bottom of both sides of the two long walls of the structure, allowing water to flow to the pumps. Water would be pumped over the levee via pipelines located within the bridge structure required to access the intake, and would be routed to the treatment plant. As with all alternatives, the operating range of the fish screen intake structure would span from historical low flows to 100-year flood conditions.

Figure 3-14 Pier Intake and Pump Station
(Sacramento's E.A. Fairbairn Intake used as an example)

3.4.1.1.2. Alternative 2 – Pier Intake Structure with Vertical Fish Screens on Two Sides, Gravity Flow to Land-Side Pump Station

This alternative would incorporate an in-river intake structure, similar in size and orientation to Alternative 1, but with an overall height about 25 feet lower (since it would not house pumps and associated electrical gear, etc.) (see **Figure 3-15**). As with Alternative 1, vertical flat panels of stainless steel wedgewire fish screens would be located at the bottom of both sides of the two long walls of the structure. Water would flow into the structure, but unlike Alternative 1, water would flow by gravity through pipes under the levee to an underground concrete, box-shaped structure (sump), which would be about 20 feet wide by 100 feet long by 10 feet tall. Pumps located directly above the sump would then be used to direct water to the treatment plant.

Figure 3-15 Pier Intake with Land-Side Pump Station
(Sacramento's E.A. Fairbairn Intake used as an example, photo modified for illustrative purposes)

3.4.1.1.3. Alternative 3 – Pier Intake Structure and Pump Station with Vertical Fish Screens on One Side

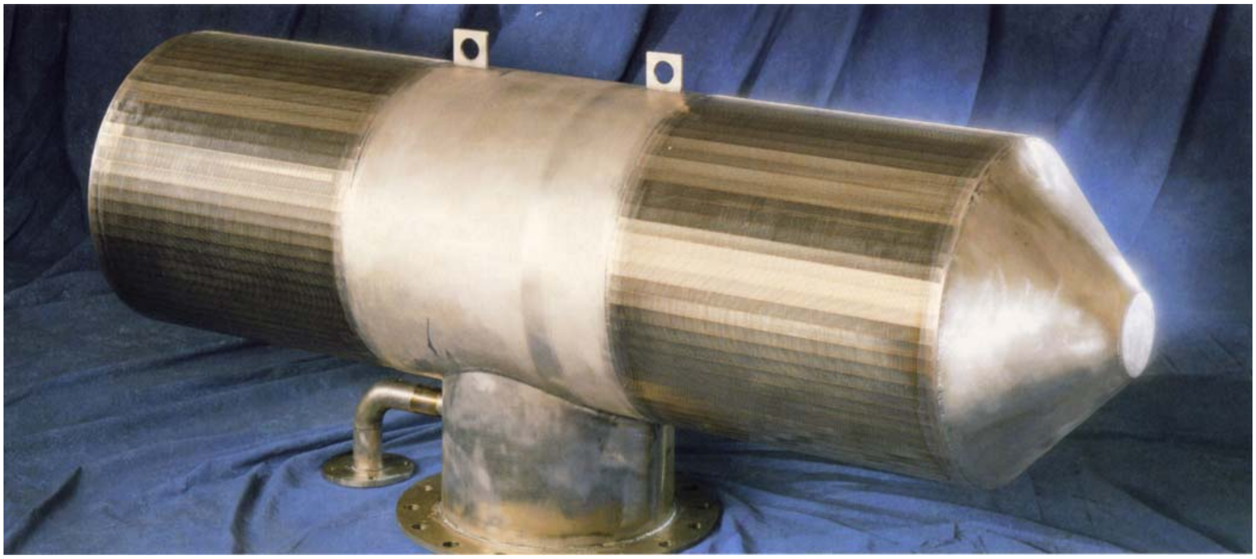
This alternative is essentially the same as Alternative 1, except that fish screens would be located at the bottom of only the river-facing long wall of the structure and not the levee-facing wall (similar to **Figure 3-14**).

3.4.1.1.4. Alternative 4 – Pier Intake Structure with Vertical Fish Screens on One Side, Gravity Flow to Land-Side Pump Station

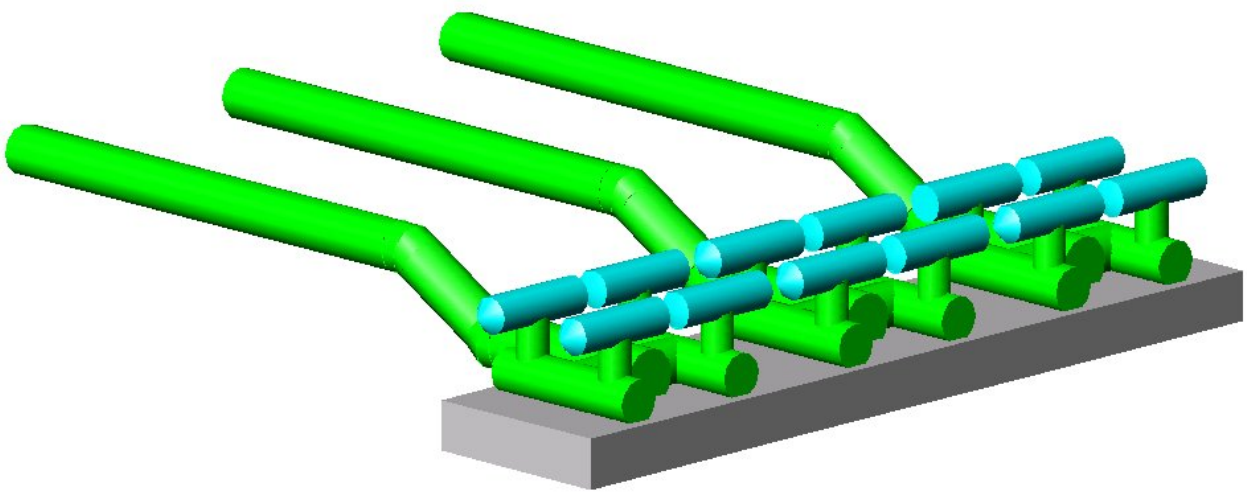
This alternative is essentially the same as Alternative 2, except that fish screens would be located at the bottom of only the river-facing long wall of the structure and not the levee-facing wall (similar to **Figure 3-15**).

3.4.1.1.5. Alternative 5 – Cylindrical Tee Screens with Land-Side Pump Station

This alternative would include a completely submerged intake structure that uses cylindrical-shaped tee screens (see **Figure 3-16**). Twelve tee screens, approximately 5 feet in diameter each and located on a concrete platform on the river bed, would be manifolded together and connected to piping routed under the levee. Water would flow by gravity through the screens and through the piping to an underground concrete sump, similar to Alternative 2. Tee screens typically use a high-energy air-burst system for cleaning, where a large volume of pressurized air would be quickly released from a land-based tank and forced through small-diameter piping into and through the screens in the reverse direction of water flow. This alternative would also include a land-side pump station similar to Alternative 2.



(a) Typical Cylindrical Tee Screen



(b) Submerged Cylindrical Tee Screen Manifold

Figure 3-16 Conceptual Plan of a Cylindrical Tee Screen

3.4.1.1.6. Alternative 6 – In-Bank Intake Structure and Pump Station with Vertical Fish Screens

This alternative is nearly the same as Alternative 3, except that the intake would be located in the bank of the river rather than out in the river (see **Figure 3-17**). However, unlike Alternative 3, this alternative would require sheet pile flow training walls upstream and downstream of the intake structure to optimize hydraulic flow conditions.

Figure 3-17 In-Bank Intake and Pump Station (Sacramento's E.A. Fairbairn Intake used as an example, photo modified for illustrative purposes)

3.4.1.1.7. Alternative 7 – In-Bank Intake Structure and Pump Station with Inclined Fish Screens

This alternative is essentially the same as Alternative 6, except that the intake screens would be "layed back" at an angle at or near the angle of the bank rather than oriented vertically (similar to **Figure 3-17**).

3.4.1.1.8. Alternative 8 – Floating Barge with Coanda Screens and Pump Station.

This alternative would incorporate a floating barge or dock, about 15 to 20 feet wide by 300 to 400 feet in length, that would adjust to the water surface elevation by sliding up and down on cylindrical steel piles set in the river. Inside the floating barge, the system would include on ogee-shaped (a flattened S-shape similar to a pool slide) coanda wedgewire fish screen. The partially submerged barge would allow water to flow over the coanda screen where any fish and solids would be screened out. This bypass flow would be pumped from the inside of the barge back to the river using a Wemco-type pump, which does not injure or kill fish. Screened water would be pumped over the levee and to the treatment plant via two to three 84- to 96-inch-diameter flexible pipes, anchored in some fashion along an access bridge provided for screen maintenance.

3.4.1.1.9. Alternative 9 - Screw Pump to Land-Side Fish Screens and Pump Station

This alternative would use three to four unscreened, inclined, 10- to 12-foot-diameter, Archimedes-type pumps in the river to lift water over the levee and into a roughly 100-foot by 60-foot land-side fish screen structure and pump station. Archimedes units use a slowly rotating screw (auger) bonded within a metal cylinder to gently lift water and fish. Fish and bypass flow would be pumped or gravity-fed back into the river while screened water would be pumped to the treatment plant. An access bridge would be constructed for maintenance of the screw pumps and a required trash rack at the mouth of the screw pumps.

3.4.1.1.10. Alternative 10 - Concrete Culvert Through Levee with Land-Side Fish Screens and Pump Station

This alternative would incorporate a concrete culvert, about 12 feet by 12 feet in dimension, extending through the levee. Water would flow by gravity through the culvert to a land-side fish screen structure and pump station, similar to Alternative 9. Fish and bypass flow would be pumped back to the river using a Wemco-type pump, and screened water would be pumped to the treatment plant. An access bridge would be constructed for maintenance of a required trash rack at the mouth of the culvert.

3.4.1.1.11. Alternative 11- Ranney Collectors with In-Bank Pump Station

This alternative would include a minimum of five to six large-diameter caissons (buried vertical, concrete, pipe-shaped structures, about 24 feet in diameter) equally spaced in the bank on the river-side of the levee. Each caisson would have perforated collector piping extending, from near its base, under the riverbed in a horizontal direction. A manifold system would collect all the water into one of the caissons, which would also include a pump station. Water would be pumped over the levee and to the treatment plant via pipes integral within the pump station's access bridge.

3.4.1.2. Intake Criteria Development

Intake design criteria were first "brainstormed" and then refined. Criteria included fish protection, lower potential for damage from river debris, lower potential for levee disturbance, lower relative first cost, lower relative operation and maintenance cost, water supply reliability, lower potential for environmental impacts, technical feasibility, public safety, and security. The following paragraphs describe the criteria and key factors that caused alternatives to score well (high score) or poorly (low score) for each intake design criterion.

3.4.1.2.1. Fish Protection

Rated the potential of an alternative to ensure that fish would not be injured or killed during their separation from diverted flow. Higher (better) scores were assigned to alternatives that would not involve bypass systems that could potentially injure, disorient, or kill fish. Also, systems with shorter screen lengths scored higher due to the shorter time fish would be exposed to potentially harmful screen-induced currents.

3.4.1.2.2. Lower Potential for Damage from River Debris

Rated the potential of the fish screens, cleaning system, and structural components to avoid impacts and damage from floating or submerged objects. Higher scores were assigned to alternatives that would have fewer components in the river. Lower scores were assigned to alternatives with more, or more vulnerable, components in the river.

3.4.1.2.3. Lower Potential for Levee Disturbance

Rated the potential of the alternative to avoid physical disruption to the levee during construction of the intake structure and pump station. An alternative scored higher if minimal permanent and/or temporary levee disturbances and modifications would occur. For example, an alternative would score higher if it included pumping water over the levee rather than if it included trenching through the levee for a gravity-flow system to land-side pumps.

3.4.1.2.4. Lower Relative First Cost

Rated the potential of the alternative for lower cost of construction. An alternative scored higher if the estimated construction cost was lower relative to other alternatives.

3.4.1.2.5. Lower Relative Operation and Maintenance Cost

Rated the potential of the alternative for lower estimated cost of operation and maintenance for the intake structure, fish screens, pump station, bypass pumps, and other associated features. An alternative scored higher if the facility would be integrated into a single structure or building, and if it would involve fewer mechanical, electrical, and structural components. A higher score was also assigned to an alternative that would require fewer operators. An alternative that would require more periodic expert maintenance was assigned a lower score for this criterion.

3.4.1.2.6. Water Supply Reliability

Rated the ability of the alternative to consistently provide the desired flow. An alternative that would use proven technology received a higher score. An alternative received a lower score if it had more components that could potentially break down and/or could require a long period of time to repair.

3.4.1.2.7. Lower Potential for Environmental Impacts

Rated the potential of the alternative to avoid environmental impacts during both construction and operation and maintenance. The following subcriteria were considered in this evaluation: aesthetics, biological resources, noise, recreation, traffic, and hydrology. An alternative scored high if minimal disturbance would occur during construction and operation, and if the proposed alternative would have a relatively small size, or footprint.

3.4.1.2.8. Technical Feasibility

Rated the perceived design and construction difficulty of the alternative. An alternative similar in design to one known to have been constructed and successfully operated scored higher than an unproven design. In addition, an alternative that would require uncommon materials or equipment and/or atypical or unproven design or construction techniques scored lower.

3.4.1.2.9. Public Safety

The proposed intake would be located in an area used for sports fishing, boating, and other recreational activities. This criterion rated the degree to which the safety of the general public using this reach of the river could be impacted. An alternative scored higher if fewer elements of the alternative were in the river, thereby reducing the potential for an incident involving the public. An alternative scored lower if it had in-river facilities that were difficult to monitor and/or presented an “attractive nuisance.”

3.4.1.2.10. Security

Rated the degree to which the alternative would potentially be exposed to vandalism or terrorism. An alternative scored higher if it would have fewer components exposed or accessible, or if public access could be more easily controlled. For example, cylindrical tee screens would be completely submerged deep in the river and relatively inaccessible, so an alternative using these screens scored higher.

3.4.1.3. Weighting of Criteria

Criteria were weighted based on the consensus of workshop members regarding the relative importance of the criteria to the project. Each criterion was assigned a relative weight as a percentage, with the total for all criteria summing to 100 percent. The following bullet items summarize the rationale used to assign relative weights for each criterion. Weightings also are summarized in **Table 3-2**.

- Water supply reliability was assigned the highest relative weight of 15 percent because as a municipal and industrial (M&I) water source, it is fundamental that the alternative provide reliable water at all times.
- Lower relative first cost, technical feasibility, and fish protection were each assigned a 13 percent relative weight. The cost of the facility will obviously be a key factor for the partners, and both technical feasibility and fish protection are key aspects in successfully designing, constructing, and obtaining regulatory permits for the facility.
- Public safety and lower relative operation and maintenance cost were each assigned a 10 percent relative weight. Public safety is a key issue for all public agencies and potential liability resulting from persons accessing the facility and being injured is an important consideration. Lower operation and maintenance costs were also considered important.
- Both lower potential for environmental impacts and security were assigned a relative weight of 8 percent. While very important to the project, these criteria were considered slightly less important than public safety. It should be noted that it is the intent of this evaluation to be sensitive to environmental concerns and that a detailed, in-depth environmental assessment will be conducted subsequent to this initial evaluation.
- Lower potential for damage from river debris, assigned a relative weight of 6 percent, is an important consideration when designing an in-river structure but was considered less important than lower potential for environmental impacts and significantly less important than public safety.
- Lower potential for levee disturbance was assigned the lowest relative weight of 4 percent. Levee disturbance is an important issue relative to the difficulty of construction and the ability to obtain a Reclamation Board permit. However, this issue was considered less important than reducing potential damage to the structure from river debris and significantly less important than lower operation and maintenance costs.

Table 3-2 Criteria Weights for Intake Structure and Fish Screen Initial Screening Process

No.	Criteria	Criteria Weight (percent)
1	Fish Protection	13
2	Lower Potential for Damage from River Debris	6
3	Lower Potential for Levee Disturbance	4
4	Lower Relative First Cost	13
5	Lower Relative Operation and Maintenance Cost	10
6	Water Supply Reliability	15
7	Lower Potential for Environmental Impacts	8
8	Technical Feasibility	13
9	Public Safety	10
10	Security	8
	Total	**100**

3.4.1.4. Rating of Alternatives

Each of the eleven alternatives was scored on a scale of 1 to 5 for its ability to meet each of the 10 criteria. A score of 5 meant that an alternative had the best ability to meet the criterion, a score of 1 meant the alternative was least successful in meeting the criterion, and a score of 3 meant the alternative had an average/good ability to meet the criterion. The alternative's score for each criterion was multiplied by the criterion's weighting factor and the products were summed to obtain an overall score for each alternative. The highest (best) possible overall score was 5 and the lowest (worst) possible score was 1.

Results of the evaluation of alternatives are presented in **Table 3-3**, where alternatives are arranged from highest score to lowest score. The advantages and disadvantages of each alternative are summarized below.

Table 3-3 Initial Screening of Intake Alternatives for the Proposed Elverta Site

ALTERNATIVES	CRITERIA (WEIGHTING FACTOR)										
	Fish Protection (13%)	Lower Potential for Damage from River Debris (6%)	Lower potential for Levee Disturbance (4%)	Lower Relative First Cost (13%)	Lower Relative O&M Cost (10%)	Water Supply Reliability (15%)	Lower Potential for Environmental Impacts (8%)	Technical Feasibility (8%)	Public Safety (10%)	Security (8%)	Total Weighted Score
Alternative 6- In-bank intake structure and pump station with vertical fish screens In-bank intake structure/pump station, vertical fish screens, flow training walls upstream and downstream of intake structure, pump raw water over levee via access bridge	3	3	5	3.5	3	4	3	5	3.5	4	**3.69**
Alternative 1 - Pier intake structure and pump station with vertical fish screens on two sides Pier intake structure/pump station in river, vertical fish screens on two sides, pump raw water over levee via access bridge	4	2	5	3	3	4	4	4	3	3	**3.51**
Alternative 7 - In-bank intake structure and pump station with inclined fish screens In-bank intake structure/pump station, fish screens inclined to bank angle, flow training walls upstream and downstream of intake structure, pump raw water over levee via access bridge	3	4	5	3	3	4	3	2	4	4	**3.34**
Alternative 3 - Pier intake structure and pump station with vertical fish screens on one side Pier intake structure/pump station in river, vertical fish screens on one side, pump raw water over levee via access bridge	3	2	5	2.5	3	4	4	4	3	3	**3.32**
Alternative 5 - Cylindrical tee screens with land-side pump station Cylindrical tee screens completely submerged in-river, gravity feed raw water under levee to land-side pump station, screen access via barge	4	1	2	4	4	3	2	4	2	5	**3.31**
Alternative 10 - Concrete culvert through levee with land-side fish screens and pump station Gravity flow under levee via concrete culvert, land-side fish screen and pump station structure, gravity bypass fish back to river, access bridge for in-river trash rack required	2	5	2	2	3	4	2	4	4	4	**3.20**
Alternative 2 - Pier intake structure with vertical fish screens on two sides, gravity flow to land-side pump station Pier intake structure in river, vertical fish screens on two sides, gravity feed raw water under levee to land-side pump station, access required for maintenance	4	2	2	2	3	4	2	4	3	4	**3.18**
Alternative 11 - Ranney collectors with in-bank pump station Ranney collector caissons (minimum 4) located in bank, perforated pipes extend under river to collect raw water, gravity flow to common pumps in one caisson, pump over levee via access bridge	5	5	5	2	2	2	2	2	5	4	**3.15**
Alternative 4 - Pier intake structure with vertical fish screens on one side, gravity flow to land-side pump station Pier intake structure in river, vertical fish screen on one side, gravity feed raw water under levee to land-side pump station, access bridge required for maintenance	3	2	2	2	3	4	2	4	3	4	**3.05**
Alternative 9 - Screw pump to land-side fish screens and pump station Archimides screw pumps water and fish over levee, land-side fish screen and pump station structure, gravity bypass fish back to river, access bridge for in-river trash rack	2	5	5	3	2	2	4	2	3.5	4	**2.90**
Alternative 8 - Floating barge with Coanda screens and pump station In-river floating barge containing coanda screens and pump station, barge attached to cylindrical steel piles, fish bypass pumped directly into river, raw water pumped over levee through large-diameter flexible piping	1	1	4	1	1	1	4	1	1	1	**1.36**

Notes:

1. Alternatives were rated for each criterion on a scale of 1 to 5, with 1 being the least able to meet the criterion and 5 being best able to meet the criterion.
2. For each alternative the weighted scores for all criteria were summed to obtain a total weighted score. The highest (best) possible weighted score was 5.00 and the lowest was 1.00.
3. The evaluation table above was developed at an all-day meeting attended by several of MWH's leading fish screened intake engineers.
4. Criteria were weighted based on the group's consensus opinion of their relative importance. For example, the ability of the facility to assure a reliable water supply was weighted higher (15%) than the expected extent of levee disturbance during construction (4%).

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3.4.1.4.1. Alternative 6 (Score 3.69/5.00) – In-Bank Intake Structure and Pump Station with Vertical Fish Screens

Alternative 6 ranked well for most criteria and earned the highest overall rank in the initial evaluation. The advantages and disadvantages of this alternative are listed below:

Advantages:

- Proven vertical fish screen technology is approved by the anadromous fish technical team as a safe screening system for the target fisheries.
- Successful intake structure configuration is similar to elements of the design of Sacramento's Sacramento River WTP Intake, E.A. Fairbairn WTP Intake, and the proposed design for Reclamation District 2035's new 400-cfs intake. The large, oblong, concrete structure is durable and safe.
- Interference with navigation and recreational activities and impact on flood conveyance would be reduced due to location of the intake structure within the river bank.
- Constructibility would be improved since location of the intake structure within the river bank allows direct accessibility from land, rather than from a barge or a temporary bridge.
- Levee disturbance and environmental footprint would be reduced by routing piping through the access bridge and over the levee rather than trenching through the levee.

Disadvantages:

- Upstream and downstream training walls would be required to smooth river streamlines to decrease fish swimming disruption. This would increase the environmental footprint of the structure relative to an in-river pier and also increase impact to shaded riverine habitat.
- Locating fish screens on only one side of the structure rather than both sides could cause the overall structure to be longer, potentially increasing cost and increasing the amount of time fish are exposed to the screens.
- Raising and regrading roughly 1,200 feet of the levee road (Garden Highway) would be required to facilitate the transition between the road and the access bridge.

3.4.1.4.2. Alternative 1 (Score 3.51/5.00) – Pier Intake Structure and Pump Station with Vertical Fish Screens on Two Sides

Alternative 1 ranked a close second to Alternative 6. The advantages and disadvantages of this alternative are listed below:

Advantages:

- Proven vertical fish screen technology is approved by the anadromous fish technical team as a safe screening system for the target fisheries.
- Successful intake structure configuration is identical to the design of Sacramento's Sacramento River WTP Intake and E.A. Fairbairn WTP Intake. The large, oblong, concrete structure is durable and safe.
- Levee disturbance and environmental footprint would be reduced by routing piping through the access bridge and over the levee rather than trenching through the levee.

- Potential would exist for a shorter effective fish screen length, and therefore improved fish protection, since the total screen length would be split between two walls of the structure rather than a single wall.
- Disruption of the riverbank would be reduced since the structure is farther out in the river.

Disadvantages:

- This alternative would be more vulnerable to higher velocity flows and faster moving debris due to its location closer to the middle of the river.
- This alternative would be more challenging and expensive to construct since the in-river location would require work from a barge or from a temporary construction bridge and would require a more complex cofferdam design. In addition, due to the increased river depth closer to its middle, longer and/or welded segments of sheet pile would be required to construct the cofferdam.
- Longer bridge length may require in-river support piers.
- Raising and regrading of roughly 1,200 feet of the levee road (Garden Highway) would be required to facilitate the transition between the road and the access bridge.

3.4.1.4.3. <u>Alternative 7 (Score 3.34/5.00) – In-Bank Intake Structure and Pump Station with Inclined Fish Screens</u>

Alternative 7 ranked third. The advantages and disadvantages of this alternative are essentially the same as Alternative 6 with the exceptions listed below:

Advantages:

- The angled orientation of the fish screens may improve the hydraulic flow characteristics of the structure by providing a less abrupt transition from the existing upstream bank angle to the screen "lay-back" angle.
- The angled orientation of the fish screens would provide more screen area per unit height of screen structure than a vertically oriented screen structure of the same length. This could potentially reduce the overall depth and length of the structure. However, NOAA Fisheries is considering new criteria that would only recognize the vertical projection of the slanted screen, negating this advantage.

Disadvantages:

- The inclined screen would complicate screen-cleaning system design and flow-approach velocity balancing, and may be considered somewhat less desirable by permitting agencies. In addition, the cleaning system would likely be air, which would adversely affect public safety.
- Removing the screen for maintenance or repair would require a crane and diver; design of a screen removal system may be more complex because of the inclined orientation of the screens. Although the screen removal system for Alternative 6 may ultimately require a crane and diver, the vertical orientation would allow more design flexibility and the potential that the screen might be able to slide up and out without the aid of a diver.

3.4.1.4.4. Alternative 3 (Score 3.32/5.00) – Pier Intake Structure and Pump Station with Vertical Fish Screens on One Side, Gravity Flow to Land-Side Pump Station

Alternative 3 ranked fourth. The advantages and disadvantages of this alternative are essentially the same as Alternative 1 with the exceptions listed below:

Advantages:

- Having fish screens on only one side of the structure would reduce the complexity of the screen cleaning system and potentially the screen removal system. The reduction in complexity would simplify the design and maintenance of both the cleaning system and the fish screens.

Disadvantages:

- Locating fish screens on only one side of the structure rather than both sides could cause the overall structure to be longer, potentially increasing cost and increasing the amount of time fish are exposed to the screens.

3.4.1.4.5. Alternative 5 (Score 3.31/5.00) – Cylindrical Tee Screens with Land-Side Pump Station

Alternative 5 ranked fifth. The advantages and disadvantages of this alternative are listed below:

Advantages:

- Proven cylindrical tee screen technology is approved by the anadromous fish technical team as a safe screening system for the target fisheries.
- Successful intake structure configuration is similar to the 150 cfs M&T/Parrott intake on the Sacramento River near Chico.
- Cylindrical tee screens would offer a significantly reduced structure on the river and associated reduction in first cost.
- Operation and maintenance difficulty and costs would be reduced since the design has no moving parts in the river.
- The submerged configuration of the screens would afford better security for the facility, as would the more easily monitored land-side pump station.

Disadvantages:

- The screens' exposure to submerged or partially submerged river debris, such as trees and logs, makes them more susceptible to damage.
- The pipeline carrying water from the intake to the land-side pump station would be trenched under and through the levee. An excavation of the depth and width required to install the pipe under the levee (up to 44 feet deep and 50 to 200 feet wide) would significantly increase the footprint of environmental disturbance for the project.
- Access to the screens would be limited. If an individual screen were damaged, divers and a barge with a crane would need to be mobilized for repairs. In addition, it would be more difficult to temporarily close off or isolate a damaged screen as compared to a flat-vertical screen system with a blank steel plate that could be positioned over the opening. These characteristics would

reduce system reliability. Routine annual inspection of the screens would also require divers in the deep, fast-moving water and would be relatively expensive.

- It could be difficult to isolate boaters and jet skiers from the area above the submerged intake. Public safety issues may arise if recreational water users were above the screens when the high-energy air–burst system was activated, or if a boat anchor accidentally snagged a screen.
- Although cylindrical tee screens are a proven technology, they have typically been used on smaller diversions about half the size of the proposed diversion. New, "scale-up" problems not previously encountered could occur.
- The majority, if not all, of the tee screen river installations have been for agricultural facilities. These facilities have greater flexibility in diversion rate and can more easily schedule "down-time" if screens are damaged. This is not the case for a municipal facility.

3.4.1.4.6. <u>Alternative 10 (Score 3.20/5.00) - Concrete Culvert Through Levee with Land-Side Fish Screens and Pump Station</u>

Alternative 10 ranked sixth. The advantages and disadvantages of this alternative are listed below:

Advantage:

- Vulnerable and maintenance-intensive facility components are moved out of the river to a protected land-side location.

Disadvantages:

- Unlike the cylindrical tee screen configuration of Alternative 5, an access bridge, with associated cost, would still be required for maintaining the trash rack located at the entrance of the culvert through the levee.
- The conduit carrying water from the intake to the land-side fish screen and pump station would be trenched under and through the levee. An excavation of the depth and width required to install the conduit under the levee would significantly increase the footprint of environmental disturbance for the project.
- Fish protection would be reduced since the fish, along with a permit-required quantity of bypass flow, would either be pumped or would gravity-flow through piping back to the river.
- The required bypass system would also increase the initial facility cost, operation and maintenance costs and complexity, and regulatory scrutiny and reporting requirements.

3.4.1.4.7. <u>Alternative 2 (Score 3.18/5.00) – Pier Intake Structure with Vertical Fish Screens on Two Sides, Gravity Flow to Land-Side Pump Station</u>

Alternative 2 ranked seventh. This alternative is similar to Alternative 1 with respect to intake structure shape and fish screen orientation, but is similar to Alternative 10 regarding gravity flow through the levee and pump location. The advantages and disadvantages of this alternative are listed below:

Advantages:

- Proven vertical fish screen technology is approved by the anadromous fish technical team as a safe screening system for the target fisheries.

- Successful intake structure configuration is similar to the design of Sacramento's Sacramento River WTP Intake and E.A. Fairbairn WTP Intake. The large, oblong, concrete structure is durable and safe.
- Potential exists for a shorter effective fish screen length, and therefore improved fish protection, since the total screen length is split between two walls of the structure rather than a single wall.

Disadvantages:

- This alternative is more vulnerable to higher velocity flows and faster moving debris due to its location closer to the middle of the river.
- This alternative would be more challenging and expensive to construct since the in-river location would require work from a barge or from a temporary construction bridge and would require a more complex cofferdam design. In addition, due to the increased river depth closer to its middle, longer and/or welded segments of sheet pile would be required to construct the cofferdam.
- Longer bridge length may require in-river support piers.
- Raising and regrading of roughly 1,200 feet of the levee road (Garden Highway) would be required to facilitate the transition between the road and the access bridge.
- The conduit carrying water from the intake to the land-side pump station would be trenched under and through the levee. An excavation of the depth and width required to install the conduit under the levee would significantly increase the footprint of environmental disturbance for the project.
- Maintenance of mechanical equipment at two locations, the in-river intake and the land-side pump station, would be required.
- Since a land-side pump station would be required in addition to the large, expensive in-river intake, initial facility cost would increase.

3.4.1.4.8. Alternative 11 (Score 3.15/5.00) - Ranney Collectors with In-Bank Pump Station

Alternative 11 ranked eighth. The advantages and disadvantages of this alternative are listed below:

Advantages:

- This alternative would have very few accessible or vulnerable components in either the river or on the protected side of the levee.
- Levee disturbance would be reduced by routing piping through the short access bridge and over the levee rather than trenching through the levee.

Disadvantages:

- The caissons and perforated under-river piping would be technically challenging to design at the scale required for the proposed project. The concept has been successfully used but only on facilities one-half to one-quarter of the size of the proposed facility.
- The large caissons would be challenging and expensive to construct due to the excavation required and the fabrication and positioning of the large circular sections of concrete.
- The tunneling required for the numerous under-river collector pipes would be expensive.

- The flow rate to the collector pipes would be difficult to predict and could change over time due to fine particle migration and resulting clogging.
- Construction of the caissons in the sensitive riparian environment on the river-side of the levee would have increased negative environmental impacts relative to other alternatives.

3.4.1.4.9. Alternative 4 (Score 3.05/5.00) – Pier Intake Structure with Vertical Fish Screens on One Side, Gravity Flow to Land-Side Pump Station

Alternative 4 ranked ninth. The advantages and disadvantages of this alternative are essentially the same as Alternative 2 with exceptions listed below:

Advantage:

- Having fish screens on only one side of the structure would reduce the complexity of the screen cleaning system and potentially the screen removal system. The reduction in complexity would simplify the design and maintenance of both the cleaning system and the fish screens.

Disadvantage:

- Locating fish screens on only one side of the structure rather than both sides could cause the overall structure to be longer, potentially increasing cost and increasing the amount of time fish are exposed to the screens.

3.4.1.4.10. Alternative 9 (Score 2.90/5.00) – Screw Pump to Land-Side Fish Screen and Pump Station

Alternative 9 ranked tenth. The advantages and disadvantages of this alternative are listed below:

Advantage:

- This alternative could potentially meet minimum project criteria.

Disadvantages:

- Large, 10- to 12-foot diameter rotating cylinders could create an “attractive nuisance” on the river, negatively impacting public safety and facility security.
- An access bridge, with associated cost, would be required for maintaining the pumps and the trash racks located at the pump inlets.
- Fish protection would be reduced since fish would be pumped twice: once in the screw pump and again after screening in the land-side facility.
- Operation and maintenance costs would increase since the water would be pumped twice: once in the screw pumps to get over the levee, and again to the treatment plant using a different set of pumps.
- Operation costs would increase due to the requirement that the screw pumps would have to pump 5 to 10 percent more than the desired flow to provide water to pump fish back to the river.

- The technical feasibility of using screw pumps in this application is in question due to wide variation in river levels (head conditions) relative to the effective flow range of this type of pump.
- The screw pumps would be very susceptible to damage from floating debris.

3.4.1.4.11. <u>Alternative 8 (Score 1.36/5.00) – Floating Barge with Coanda Screens and Pump Station</u>

Alternative 8 ranked last. This alternative ranked well for environmental impacts, since it would not significantly disturb the bank, levee, or surrounding areas. However, the alternative rated poorly for all other criteria. The advantages and disadvantages of this alternative are listed below:

Advantage:

- Environmental footprint would be reduced in the sensitive riparian zone.

Disadvantages:

- Coanda fish screening technology is not currently approved by the governing regulatory agencies.
- Pumping fish would be required, which is less attractive to regulatory agencies.
- The large barge required would be difficult to secure from the public and could create an "attractive nuisance" on the river, negatively impacting public safety. The ease of access by recreational boaters for vandalism and accidental injury would be a significant concern.
- The barge and flexible transmission pipes would be very susceptible to damage from floating debris.
- Costs are expected to be high due to the unique "one-time" nature of the system components.
- The technical feasibility of using large-diameter flexible piping to pump continuously at the design flow rate and for this novel in-river application is in question due to material properties and availability of product in the marketplace.
- The complex pumping control system required to maintain barge depth while simultaneously diverting flow and bypass pumping fish is expected to be expensive and could jeopardize the reliability of the facility.

3.4.1.5. Conclusions of the Initial Intake Configuration Evaluation

The alternatives presented in the previous section include the full range of potential configurations for a fish-screened intake facility at the proposed site on the Sacramento River. It is clear from the ranking and by inspection of the advantages and disadvantages that certain intake configurations are obviously infeasible for the intake site and can be eliminated from further consideration.

Alternatives that can be directly eliminated include **Alternative 8 – Floating Barge with Coanda Screens and Pump Station, Alternative 9 – Screw Pump to Land-Side Fish Screens and Pump Station,** and **Alternative 11 – Ranney Collectors with In-Bank Pump Station**. These alternatives all have significant technical drawbacks. In addition, even if all of the technical difficulties could be successfully overcome, designs would be unique and unproven. Therefore, regulatory agencies would likely require extensive testing and monitoring and ultimately might not grant a permit for construction until the designs were pilot-tested at a smaller scale. While advanced design and testing of these alternatives is certainly possible, little benefit would accrue since there are no perceived cost savings, and better-suited, proven configuration alternatives are available.

Alternatives 2 and 4 – Pier Intake Structure with Vertical Fish Screens on Two Sides and One Side, respectively**, Gravity Flow to Land-Side Pump Station** can also be removed from further consideration. These alternatives have two fundamental flaws for the proposed project: greater cost and greater environmental impact than necessary. Greater costs would be incurred from construction of both the large, sturdy intake structure and a land-side pump station. It would be more cost-effective for the current project to simply add pumps to the intake structure and save the cost of the additional land-side pump station. This type of alternative is generally more attractive to agricultural facilities that require very little lift of the water (low head) and can save operating costs by not using energy to pump up over the levee. Since the proposed project requires a water elevation higher than the levee at the WTP, there would be no operational power savings. Greater environmental impacts would be caused by the excavation required to trench a gravity conduit through the levee. The trench through the levee would be up to 44 feet deep. Excavation at this depth would require a width of 50 to 200 feet, depending on the contractor’s methods, for a length of approximately 320 feet. The excavation footprint could be significantly reduced by routing the required conduit through the proposed access bridge structure above the levee and moving water via pumps relocated to the intake structure, in a configuration similar to Alternatives 1 and 6.

Alternative 10 – Concrete Culvert Through Levee with Land-Side Fish Screens and Pump Station can also be eliminated from further consideration. This “through levee” alternative has the same large environmental footprint as Alternatives 2 and 4 and is less “fish friendly.” Pumping fish has significantly more impact than screening fish out as they swim by an intake structure. Governing agencies are not likely to permit a “fish-pumping” alternative if a reasonable “non-pumping” alternative is available. While this alternative does avoid the expensive, redundant in-river intake structure of Alternatives 2 and 4, it would have higher long-term costs due to maintenance of the fish pumping system and associated agency reporting requirements. In addition, an access bridge required for routine maintenance of the in-river trash rack at the mouth of the culvert would increase costs.

Alternative 5 – Cylindrical Tee Screens with Land-Side Pump Station would reduce the cost and complexity of in-river components of the facility. In addition, the land-based, air-burst screen cleaning system would require relatively little maintenance. Key reasons this alternative is not recommended for the proposed project are system reliability, liability, and environmental/levee impacts.

As noted previously, the screens are vulnerable to damage by submerged and partially submerged river debris. Although the screens can be replaced, the time required to mobilize a diver and barge with a crane could be several days. Typical manifolding of the tee screens makes isolating a defective screen from the functional screens difficult, and regulatory agencies may require a significant reduction in diversion rate during the days required to replace the screens.

Although the screens are located below the water surface, the area above the screens must be isolated from recreational river users. Screens can be damaged by boat anchors, and the air-burst cleaning system creates a large eruption of air that rises 2 to 4 feet out of the water and could potentially destabilize a boat or jet ski. While it is possible to separate this area by buoys, it would be difficult to police the area and liability from boater injury could be significant.

The environmental impacts associated with trenching through the levee for the conduit to the land-side pump station were noted in the discussion of Alternatives 2 and 4 above. As also previously stated, energy cost savings associated with a gravity flow under the levee would not apply to the proposed project and impacts of trenching activities could be avoided under other more suitable alternatives. In-river cylindrical screens could be a cost-effective solution for a flexible, agricultural-oriented application but are not recommended for the proposed project where reliability of drinking water supply limits flexibility of operation.

Alternatives 1 and 3 – Pier Intake Structure and Pump Station with Vertical Fish Screens on Two Sides and **on One Side,** respectively, are relatively minor variations of the same concept. Neither alternative has a significant characteristic or component that makes it unsuitable, and either could be used for the proposed project. Typically, it would be assumed that having screens on only one side would necessitate a longer structure to achieve the required minimum screen area, making the alternative less desirable. However, due to the unusually deep water available at the proposed project site, differentiating the relative advantages and disadvantages of these two configurations would require a more detailed design layout and both alternatives were retained for further consideration.

Alternatives 6 and 7 – In-Bank Intake Structure and Pump Station with Vertical Fish Screens and **with Inclined Fish Screens,** respectively, are also relatively minor variations of the same concept. While both configurations could be used at the proposed project site, the vertically oriented screens of Alternative 6 are superior to the inclined screens of Alternative 7. Inclined screens have a distinct advantage at locations where little water depth is available since they provide more effective screen area per unit height and can therefore reduce the overall length of screen, and structure, required. This could be a significant cost savings in low water conditions. However, ample available water depth exists at the proposed site negating this benefit and, as previously noted, proposed new NOAA design criteria may only recognize the vertical projection of the slanted screen. In addition, cleaning methods for inclined screens are limited to high pressure water jets or the relatively unproven forced air jets. Vertical screens can also be cleaned by the proven effective traveling brush cleaning system. In addition, the vertical screen orientation would more readily accommodate the flow baffle (adjustment) system required to accurately balance the flow across the entire surface of the screen to avoid “hot spots” that could negatively affect fish. For the above reasons, Alternative 7 was eliminated and Alternative 6 was retained for further consideration.

3.4.2. Comparison of Final Alternatives and Selection of Preferred Alternative

Plan and section drawings of each of the three remaining alternatives are presented in **Figures 3-18** through **3-20**. These concept-level layouts were used to compare the relative dimensions and relative advantages and disadvantages of the alternatives. The layouts were developed using the design criteria presented in **Section 3.1** and Hydraulic Institute standards for pump intake design. Approximate pump suction bell diameters were determined based on flow rates and were used to develop facility dimensions based on Hydraulic Institute dimensional requirements. Conservative average values were used to facilitate comparison of alternatives. It was assumed that dividing walls would be placed between pumps to reduce potential disruptive flow patterns while minimizing the separation distance required between the pumps. The intake could be designed without dividing walls but separation between pumps would need to be increased on the order of 50 percent, depending on pump flow rate. The use of fewer, larger capacity pumps may also be considered during the preliminary design phase of the project. Fish screen widths and heights provide sufficient screen area such that each bay can accommodate a 33-mgd pump, and were additionally selected based on reasonable handling size for strength and constructibility, as well as adaptability to the screen cleaning system. Final selection of optimum fish screen widths and heights will be completed during the preliminary design phase of the project, in conjunction with final selection of pump size and number.

To further differentiate the three designs, an additional evaluation criterion of Facility Design Flexibility was introduced into the comparison of alternatives. It was noted that, since this is a reconnaissance-level evaluation and relatively early in the facility design development, certain of the assumed design criteria and site conditions could change prior to construction. Specifically, it was noted the possibility of expanding the 0.2 foot per second fish screen approach velocity criterion for delta smelt protection outside of their currently defined range is being considered by the United States Fish and Wildlife Service (USFWS). In addition, modifications to levees upstream of the proposed site and the possibility of constructing setback levees upstream of the proposed site are being considered by SAFCA. The effect on intake design is not certain at this point. The approach velocity change could significantly increase the total required fish screen area for the proposed intake, while the levee modifications could create a significant increase in sediment at the project site and could potentially reduce the depth of the available water level (and associated maximum fish screen depth) at the proposed intake. Therefore, it was agreed that an estimate of the proposed alternative's ability to be modified to accommodate these and other potential changes should be included in the alternative's evaluation.

Results of the comparison of the three final intake configuration alternatives are presented in **Table 3-4**. The three alternatives compared are all viable designs for the proposed site. Alternative 1 is a proven design, with installations on both the Sacramento and American rivers, and Alternatives 3 and 6 are slight variations of that design. However, Alternative 1 appears to be the best choice for the proposed site. This alternative is slightly more complex than Alternative 3 from a mechanical perspective, but provides the greatest flexibility to accommodate potential regulatory or physical site changes, as described in the preceding paragraph. Of the three alternatives, the design of the two-sided structure could most easily accommodate either an increase in total screen area or a reduction in screen depth. The one-sided structures of the other two options, however, would require an excessively tall screen or increased facility footprint to accommodate screen area or depth changes.

In addition, Alternative 1 has a smaller footprint relative to Alternative 6, thereby causing less environmental disturbance. Also, although Alternative 6 is believed to have the lowest construction cost due to the potential for lower-risk, land-based construction, future structural analysis may reduce this benefit if a greater number of piles are required to balance seismic forces generated by the adjacent stream bank. Although Alternatives 3 and 6 would have a slightly less complex screen cleaning system since it would only be located on one side of the structure, this advantage is offset by the proven design record and greater design flexibility of Alternative 1.

Based on the advantages discussed above and shown in **Table 3-4**, Alternative 1 **-** Pier Intake Structure and Pump Station with Vertical Fish Screens on Two Sides will be retained as the preferred intake configuration for the SRWRS Elverta Diversion Alternative.

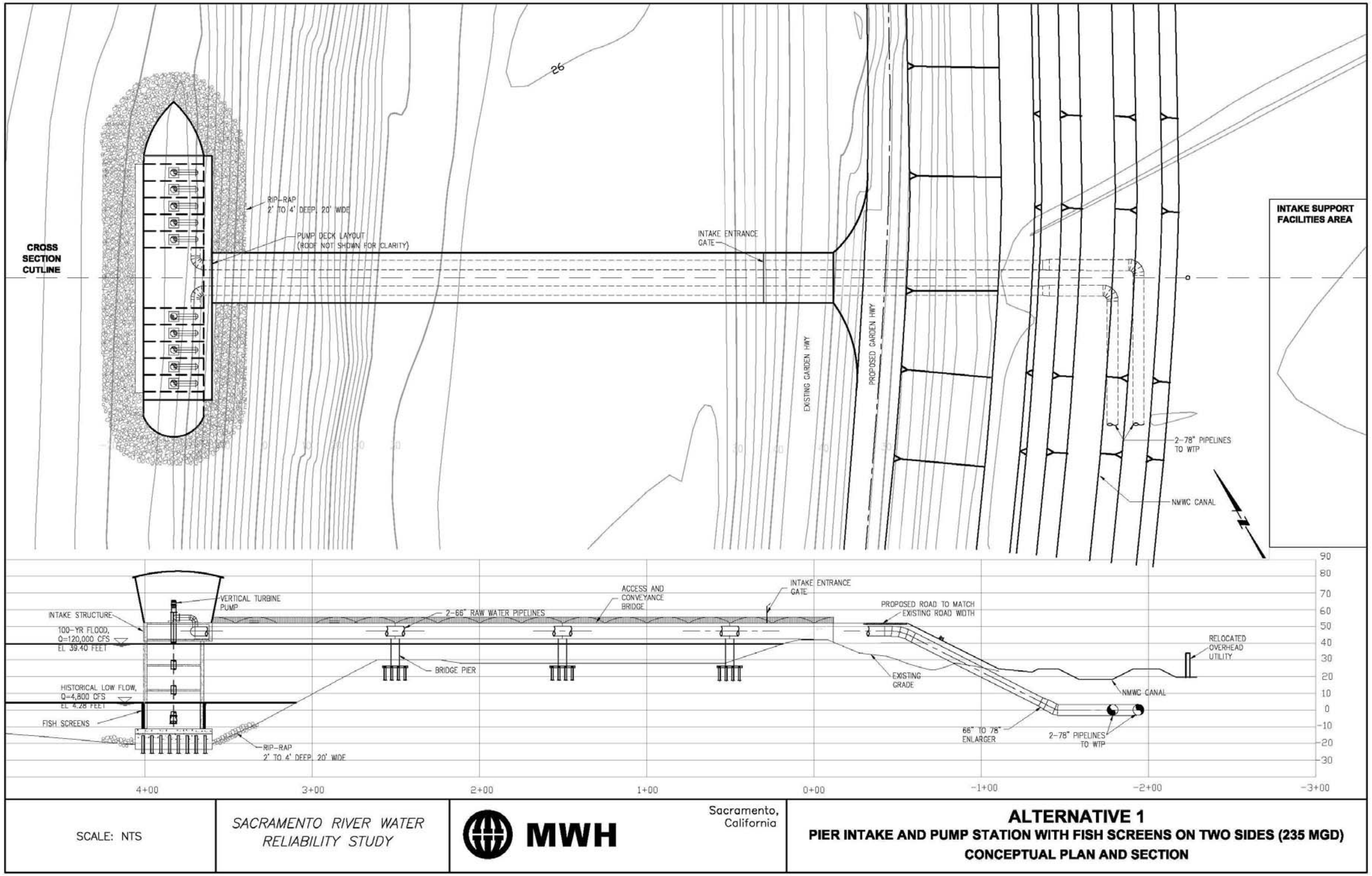

Figure 3-18 Alternative 1 – Pier Intake Structure and Pump Station with Vertical Fish Screens on Two Sides

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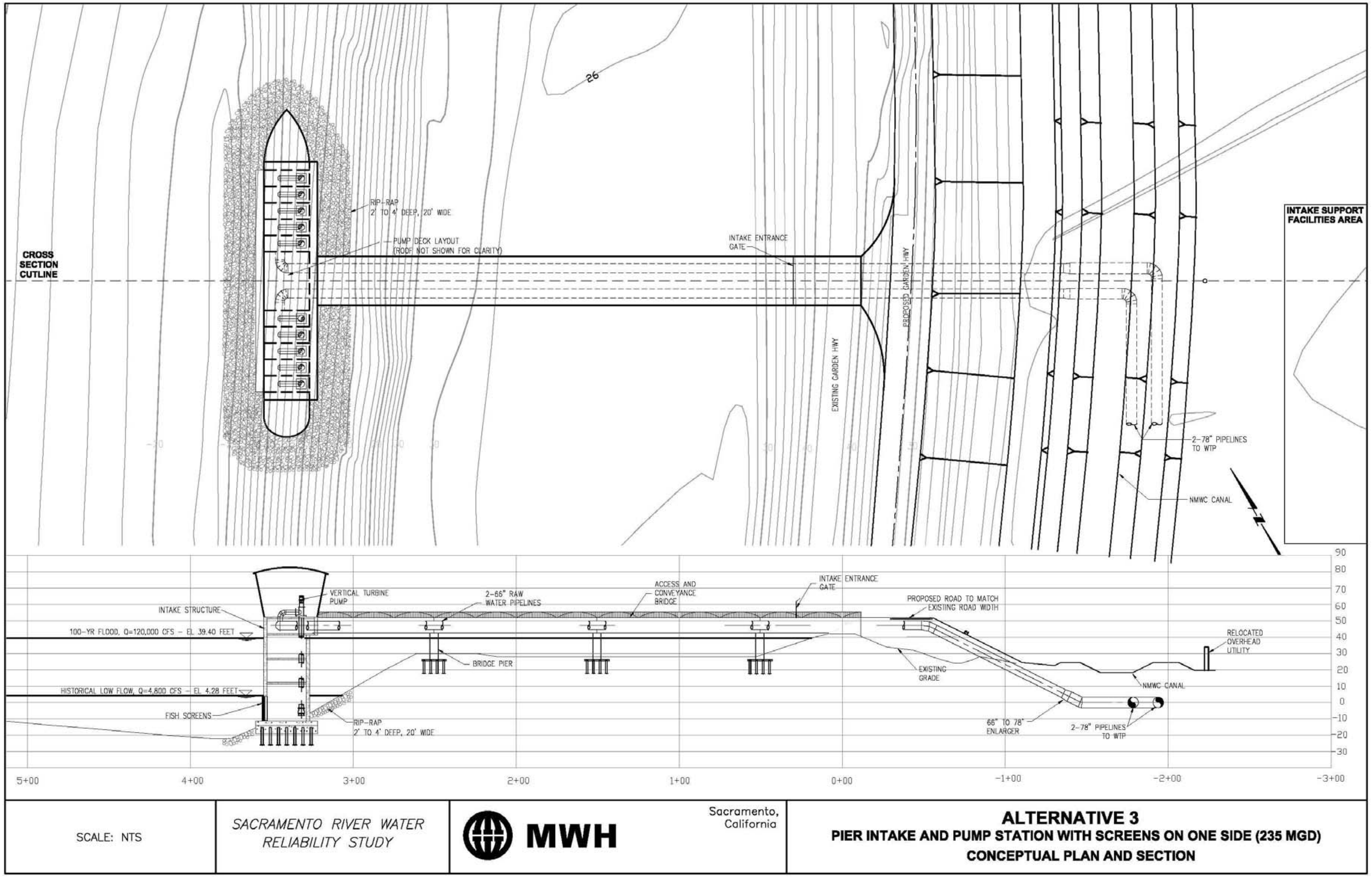

Figure 3-19 Alternative 3 – Pier Intake Structure and Pump Station with Vertical Screens on One Side

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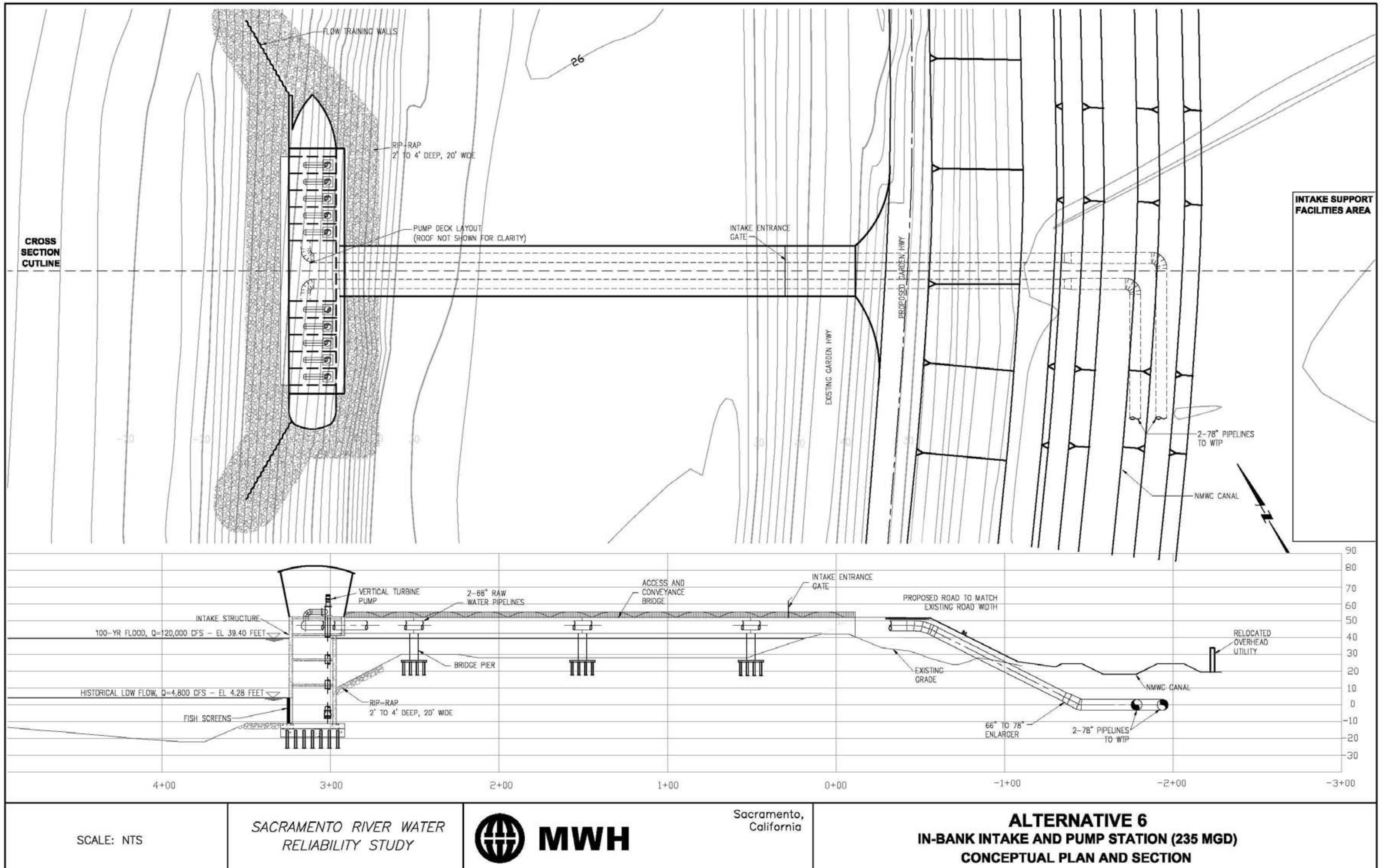

Figure 3-20 Alternative 6 – In-Bank Intake Structure and Pump Station with Vertical Fish Screens

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Table 3-4 Comparison of Final Intake Configuration Alternatives

Comparison Item	Alternative 1: Two-Sided Pier	Alternative 3: One-Sided Pier	Alternative 6: In-Bank
Intake Structure Dimensions (feet)	Length (at base) = 200 Width (at base) = 36	Length (at base) = 200 Width (at base) = 28	Length (at base) = 200 Width (at base) = 28
Fish Screen Quantity and Average Dimensions (feet)	20 Screens Width = 8 Height = 10	10 Screens Width = 8 Height = 20	10 Screens Width = 8 Height = 20
Length of Bridge (feet)	375	325	305
Overall Intake Environmental Footprint (acres)	Area (at base, including riprap) = 0.46	Area (at base, including riprap) = 0.42	Area (at base, including riprap) = 0.51
Distance from Top of Bank to River-Side of Intake (feet)	105	65	40
Advantages	Design and construction similar to two existing Sacramento facilities Offers most flexible design to accommodate regulatory or physical site changes	Reduced bridge length Screens on one side reduce complexity of cleaning system and improve screen accessibility	Intake location may allow simpler, less expensive construction from land Reduced bridge length Screens on one side reduce complexity of cleaning system and improve screen accessibility
Disadvantages	Longest bridge More screens and screen cleaners to maintain More difficult construction relative to Alternative 6	More difficult construction relative to Alternative 6 Footprint would increase significantly to accommodate regulatory or physical site changes	Larger environmental footprint in sensitive area More in-river excavation required More complicated structural design and more piles required due to unbalanced soil loading on intake Footprint would increase significantly to accommodate regulatory or physical site changes
Estimated Relative Construction Cost	Highest	Lower	Lowest

3.5. POWER, SEWER, STORM DRAINAGE, AND SPECIAL CONSIDERATIONS AT THE PROPOSED INTAKE FACILITY

In this selection, details are discussed of power feed and supply for the intake facility required for the SRWRS Elverta Diversion Alternative. Wastewater facilities, stormwater management, and coordination with the FAA and Sacramento County Airport Service are also described. Additional or modified requirements for the Joint SRWRS-ABFSHIP Elverta Diversion Alternatives are presented in **Section 3.7**.

3.5.1. Power Feed and Supply at 235 mgd Intake Facility

Aspects of power feed and supply for this facility would include power requirements, availability, and reliability, motor starter requirements, backup options, and dual feed options.

3.5.1.1. Primary Power Requirement and Availability

The maximum power requirement for the 235 mgd Elverta Intake has been estimated to be 3,850 kilovolt-amperes (kVA). **Table 3-5** generally summarizes power requirements.

Table 3-5 Power Requirement Summary for 235 mgd Facility

Intake Pump Station	Peak Flow (mgd)	Pump Load(1) (hp)	Misc. Loads (kVA)	Power (kVA)	Amps @ 4,160 Volts	1/2 Load (kVA)
Intake Facility	235	3,600	250	3,850	530	1,925

Notes:
(1) Includes a spare pump.
Key:
hp – horsepower kVA – kilovolt-ampere mgd – million gallons per day

Power would enter the site and go directly to transformers to reduce voltage from 69 kilovolts (kV) to 4.16 kV. Power from the secondary transformers would then go to two main breakers at the Elverta Intake power distribution switchgear. The transformer area is expected to be approximately 130 feet by 130 feet per Sacramento Municipal Utility District (SMUD) requirements. The intake medium voltage switchgear building is expected to be approximately 40 feet by 40 feet. The building would house the two mains, a tie breaker, potential transformer (PT) and control power transformer (CPT), and each of the two buses at 4.16 kV with two 800-amp main breakers. The distribution switchgear would have breakers to feed all of the 4.16 kV loads at the intake. Exhaust fans and heaters would be minimum building requirements.

It is anticipated that the 480-volt loads would be distributed from one switchboard to serve the plant's 480-volt motor control center loads.

SMUD is the governing power utility for the proposed WTP sites as per Article 11, Section 9, of the California Constitution. Power for this load is available from existing SMUD lines routed westward along Elverta Road up to Power Line Road. Two 69 kV power lines (in parallel) are currently in place and SMUD is currently upgrading these lines due to increased commercial and residential development in the North Natomas area. The loads presented here can be considered as part of SMUD's upgrade.

At Power Line Road, the overhead lines turn south. It is expected that SMUD would continue the feed west with 69 kV, beyond Power Line Road, using underground lines due to the runways at Sacramento International Airport.

Underground 69 kV lines have a budget cost of $175.00/foot, excluding trenching. The 69 kV service from existing upgraded overhead lines has a budget cost of $30.00/foot, excluding poles. The owner would incur the cost of poles or trenching in addition to the charge for the lines.

3.5.1.2. Utility Reliability

SMUD can provide a design that would incorporate the level of redundancy the owner would require. SMUD can design its connection points and multiple switching configurations for the redundancy that will meet the needs and satisfaction of the owner.

3.5.1.3. Motor Starter Requirements

SMUD requires all large medium voltage and low voltage motors to be reduced voltage solid state starters to reduce the impact of the starting currents on the SMUD system.

3.5.1.4. Primary Backup Power Supply

The proposed primary means for backup power supply is installing two primary feeds in the Elverta Intake site. The reliability of power supply at the site would increase greatly with installation of these separate primary feeds into the two transformers that provide the 4.16 kV at the Elverta Intake power distribution substation. The proposed plan for the power feeds at the site is to receive one feed from each of the two existing upgraded parallel 69 kV lines into the site.

Each secondary transformer would be connected to a main circuit breaker. The two mains would be connected by a tiebreaker. Upon loss of power detected in one of the two main breakers, that main would open and after a time delay (selected by the owner), the tiebreaker would close, resuming power to the side of the bus that lost power.

3.5.1.5. Alternative Backup Power Supply Option

An alternative backup power supply option would be use of a diesel generator at the Elverta Intake site. The SRWRS partners selected a 50 percent backup generation capacity for evaluation. The required 50 percent backup generation for the 235 mgd site would require a 1,925 kVA generator. A day tank (300 gallons) and fuel storage tank are required for each generator. The generator uses approximately 125 gallons of fuel per hour at full load and would require a total of 1,000 gallons of fuel for an 8-hour time period (full load). The output power for each generator would vary with load requirements; therefore, if the load was less than 1,925 kVA, the fuel consumption would be less.

The space required for the low voltage controls, day tank, and generator would be approximately 1,200 square feet in a building with integral automatic air flow louvers and fire alarm system design. Additional space outside would be required for the fuel storage tank.

A more detailed evaluation of backup power requirements and specific loads that would be deemed critical if both main breakers into the plant were lost is strongly recommended during the next phase of analysis to optimize sizing of these generators and associated facilities.

3.5.1.6. Dual Feeds from SMUD and Another Power Utility

SMUD does not allow another utility to serve within the SMUD service area.

3.5.2. Sewer System

It is assumed that an incinerator-type toilet would be provided at the intake site.

3.5.3. Stormwater Management

Currently, no storm drainage services are located in the northwest corner of Sacramento County near the project area. It has been assumed that all stormwater on the river-side of the levee would drain naturally to the river. For the approximately 0.5 acres required for the electrical substation, and standby power building on the land-side of the levee, it is assumed that the stormwater would need to be captured and managed on site. The site would be constructed and graded to collect stormwater runoff and channel it to an on-site detention basin. This basin has been sized to meet the capacity of a 10-year storm over 5 days. The Sacramento City/County Drainage Manual indicates that the water depth of such a storm would be 5.76 inches. It has been estimated that approximately 10,500 cubic feet of water would need to be planned for in the detention basin design. It was assumed that the detention basins would be 3 feet deep to allow for evaporative drying. Therefore, a detention basin approximately 60 feet by 60 feet would be required on site. An overall area of 1 acre has been reserved for the intake support facilities area.

3.5.4. Special Considerations

The intake facility is located within the overflight zone of the Sacramento International Airport. For this reason, the design of this facility must be developed to account for safety issues identified by the Sacramento County Airport Service and the FAA. Although no direct objections to the facility have been expressed in preliminary discussions with these agencies, items for continuing coordination would be overall height of the structure, design of lighting, and verification that electrical equipment at the site would not interfere with airport equipment.

3.6. CONSTRUCTION AND OPERATION OF THE PROPOSED INTAKE FACILITY

Construction characteristics and operating characteristics of the intake facility required for the SRWRS Elverta Diversion Alternative are discussed in this section. Additional or modified requirements for the Joint SRWRS-ABFSHIP Elverta Diversion Alternative are presented in **Section 3.7**.

3.6.1. Construction Characteristics

Construction of the intake structure would require erection of a sheet pile cofferdam in the river. The cofferdam would be approximately 220 feet long and 60 feet wide. Construction of the cofferdam would require placement of sheet piles, excavation within the cofferdam area, and stabilization of the cofferdam. Steel H-piles would then be driven in the cofferdam to provide structural support for the intake structure. Next, tremie concrete seal would be poured and the work area dewatered. It is estimated that construction of the cofferdam and structural piles would take approximately 18 weeks. The contractor would likely drive the piles using a floating barge as a platform. In addition, the contractor may construct a temporary H-pile bridge from the bank to the cofferdam to facilitate construction. A discharge permit would be obtained for these construction activities.

Riprap would be placed for a distance of approximately 20 feet around the intake structure to prevent scour. Prior to placement of the stones, excavation of 2 to 4 feet of native material would be required.

Excavation would be required at each of the bridge piers and clearing would be required along the full length and width of the bridge. Steel H-piles also would be driven at each pier. A concrete pile cap would also be constructed at each pier.

Due to the required elevation of the intake access bridge, the levee road (Garden Highway) would need to be raised 8 to 10 feet. This elevation change would require regrading the road for a distance of approximately 600 feet in both the north and south directions from the access bridge. Raising the levee would result in an expansion of the levee extents to a maximum of 50 feet on its landward side for the 1,200-foot regrading length. See the “section” view of **Figure 3-18** for maximum roadway cross section. The overall time for construction is estimated to take 21 to 24 months.

Limited site grading would be required on approximately 1.0 acre of land adjacent to the levee at the intake site. This site would be used for intake support facilities, including an electrical substation and standby power equipment, as required.

Construction-related traffic (e.g., materials delivery trips, workers, etc.) would access the site from Elverta Road and Garden Highway. Disposal of excavated materials and installation of concrete would require numerous truck trips to and from the site. A traffic control plan would be prepared by the contractor and reviewed by Sacramento County to make sure traffic is safely routed by the work site.

Safety on the construction site would be the responsibility of the contractor. The contractor would have a company safety program and a job-specific safety program, administered by a project safety officer. Typical procedures would include weekly safety meetings with the construction crew and hazard analyses prepared before the beginning of each new operation. Federal Occupational Safety and Health Administration (OSHA) and California (Cal)-OSHA standards would apply for all work.

The construction contract documents would include a general stormwater pollution prevention plan (SWPPP). The construction contractor would be required to submit a specific, more detailed SWPPP. The general plan would outline minimum requirements that must be met to minimize erosion and control sediments. The general and specific SWPPPs would comply with the county sediment and erosion control ordinances. Typical best management practices that would be used include the following:

- Covering all exposed slopes and stockpiles with plastic, straw, or hydroseed
- Placing silt fences at the downstream side of all work areas
- Placing a sediment filter in each drop inlet
- Sweeping all work areas frequently
- Constructing sediment ponds in key locations
- Placing waddles or hay bales across steep, disrupted slopes
- Constructing driveways at the work site exit

3.6.2. Operating Characteristics

The Elverta Intake would operate continuously, 24 hours per day, 7 days per week, at various flow rates throughout the year. Ongoing operations and maintenance would occur. The facility is unmanned; however, it is expected that maintenance personnel would visit the site at least twice per day to confirm operation and perform minor maintenance. More advanced maintenance of the pumps and motors would be required periodically.

Routine vehicle traffic would comprise mainly full-size pick-ups driven by maintenance staff. Specialty requirements for scheduled and emergency maintenance would include heavier load trucks.

3.7. JOINT SRWRS-ABFSHIP ELVERTA DIVERSION ALTERNATIVE

This section describes the Joint SRWRS-ABFSHIP Elverta Diversion Alternative (see **Figures 1-2** and **3-21**), a subalternative for the SRWRS Elverta Diversion Alternative, includes the participation of NMWC in the project intake facility. The majority of the Joint SRWRS-ABFSHIP Elverta Diversion Alternative is identical to the SRWRS Elverta Diversion Alternative, with the exception of modifications to the size of the intake facility, modifications to NMWC's existing canals, and additional facility modifications, as described in this section.

Under the Joint SRWRS-ABFSHIP Elverta Diversion Alternative, the NMWC Elkhorn Diversion included in the CALFED-supported ABFSHIP Sankey/Elkhorn diversion alternative would be consolidated with the SRWRS Elverta Intake in a joint diversion (see **Figure 3-22**). The Elverta Intake facility capacity would be increased from 235 mgd (365 cfs) to 371 mgd (575 cfs) to accommodate the capacity of 136 mgd (210 cfs) required by NMWC. In addition, the Joint SRWRS-ABFSHIP Elverta Diversion Alternative would include improvements to approximately 1.6 miles of NMWC's existing Elkhorn Main Canal as well as associated modifications required to enable delivery of the water pumped from the new intake. The remainder of the Joint SRWRS-ABFSHIP Elverta Diversion Alternative is identical to the SRWRS Elverta Diversion Alternative. It is anticipated that all facilities, with the exception of NMWC's Elkhorn Main Canal, would be constructed, owned, and operated by the SRWRS cost-sharing partners. The proposed Elverta intake structure, raw water pipelines, and North Natomas WTP would be owned and operated by Sacramento. NMWC would continue to own and operate the Elkhorn Main Canal. The treated water pipelines delivering water to PCWA, SSWD, Roseville, and Sacramento would be owned and operated by individual purveyors. Project facilities that differ from the SRWRS Elverta Diversion Alternative are described in the subsections that follow; all other facilities would be constructed as described for the SRWRS Elverta Diversion Alternative.

3.7.1. Intake Facilities

The Joint SRWRS-ABFSHIP Elverta Diversion Alternative includes modifications to the Elverta Intake as described for the SRWRS Elverta Diversion Alternative. The modifications are described below.

3.7.1.1. Pumps, Discharge Piping, and Energy Dissipation

As previously noted for the SRWRS Elverta Diversion Alternative, water would be drawn into the intake structure by vertical turbine pumps with varying capacities. Pump configuration could include two 11-mgd, two 22-mgd, and six 33-mgd pumps, with one of the six 33-mgd pumps a redundant or backup pump. To supply water for NMWC, an additional four dedicated 33-mgd pumps (approximate capacity) would be required. The pumps could discharge to a manifold and be routed across the access bridge and over the Sacramento River levee in a dedicated 72-inch pipe, as shown in **Figure 3-11**.

The NMWC water pumped from the intake is to be routed to the Elkhorn Main Canal, which is routed parallel with the levee, at a varying distance from its land-side toe. Due to the elevation difference between the top of the levee and the Elkhorn Main Canal, the pumped water would have a significant amount of excess energy that must be dissipated in a controlled fashion. It is assumed that water from the 72-inch discharge pipe would enter the canal via a concrete structure designed to dissipate its excess energy. The concrete structure is assumed to be approximately 30 feet long by 40 feet wide and approximately 12 feet in overall height. The structure is further assumed to have an 84-inch by 84-inch sluice gate at both its north and south outlets to the canal, providing a variable release rate of water in the northward or southward directions. Specific design characteristics of the energy dissipation structure will be established during the preliminary design phase of the project.

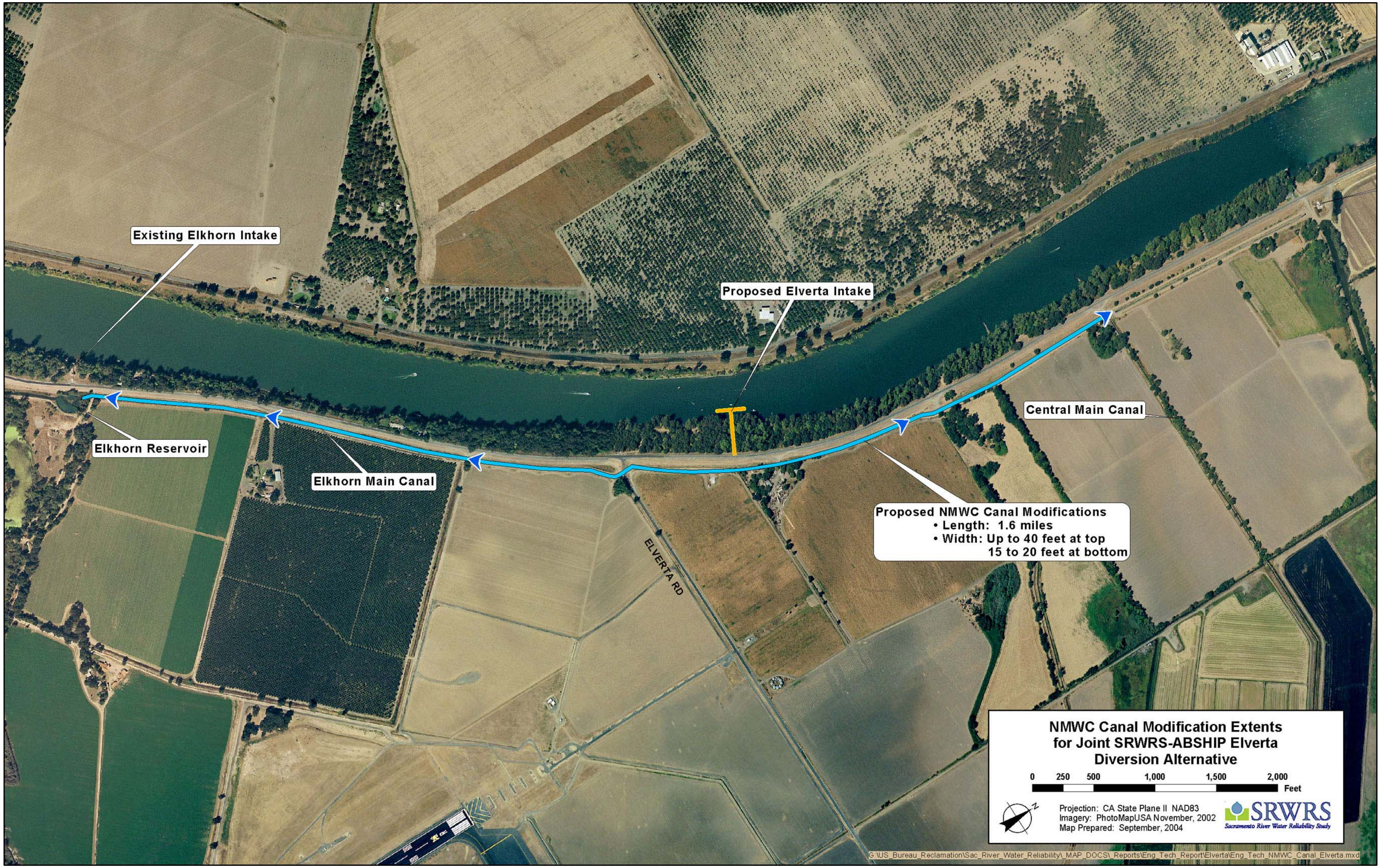

Figure 3-21 NMWC Canal Modification Extents for Joint SRWRS-ABFSHIP Elverta Diversion Alternative

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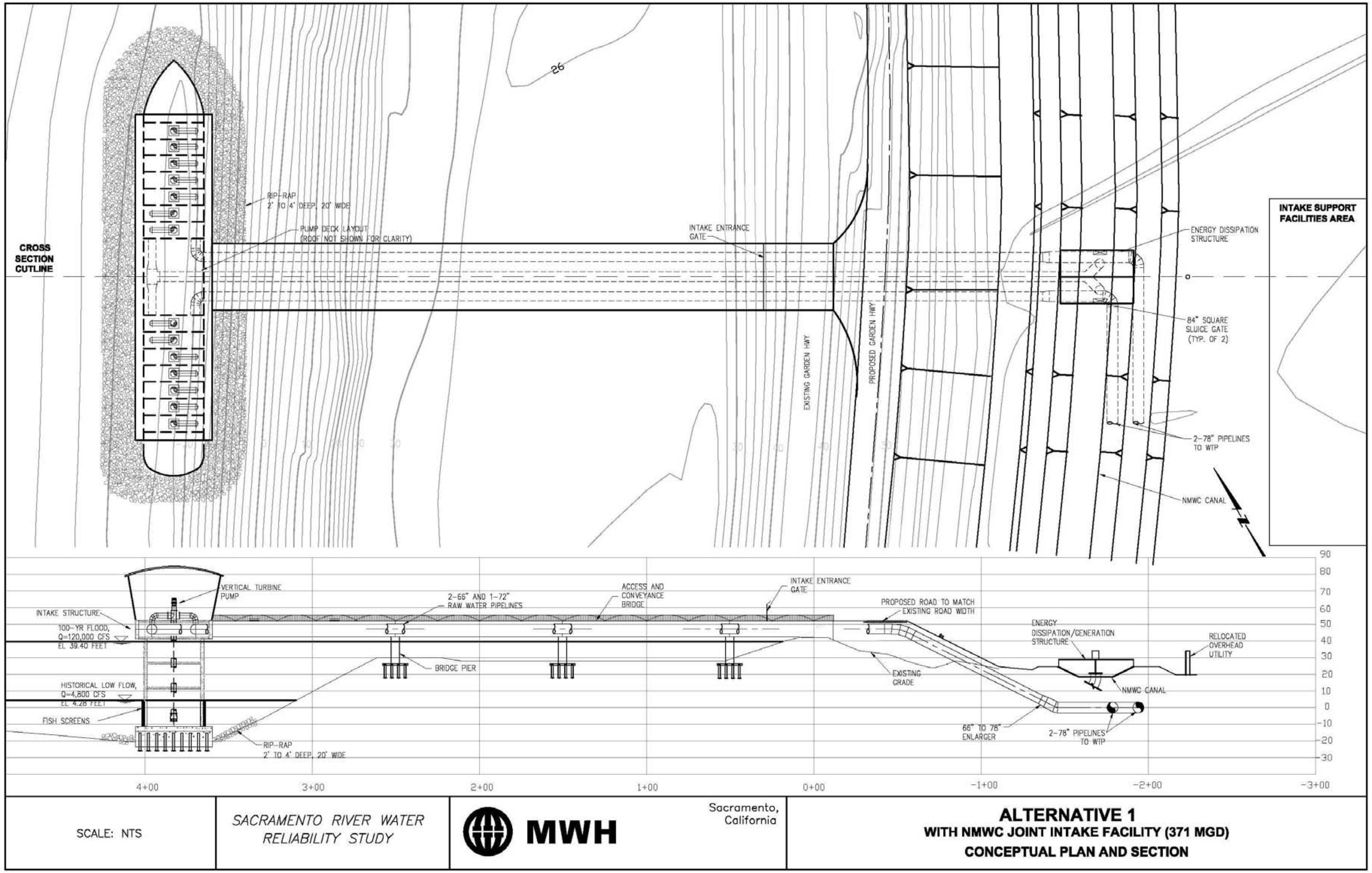

Figure 3-22 Alternative 1 with NMWC Joint Intake Facility

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3.7.1.2. Fish Screens

As previously noted for the SRWRS Elverta Diversion Alternative, the 235 mgd diversion would require 20 fish screens, each an average of 8 feet wide and 7.5 feet high. The four additional pumps required for NMWC's additional 136 mgd would require 4 additional pump bays, each requiring approximately 155 square feet of screen area divided between the two sides of each bay. If it is assumed that the additional screens would be 8 feet wide, their required height would then be approximately 10 feet each.

3.7.1.3. Intake Structure and Conveyance Bridge

The additional four pump bays required for NMWC dedicated flow would add 40 feet to the length of the joint intake structure, increasing the structure length to 240 feet. While the structure foundation width should not change from the SRWRS Elverta Diversion Alternative, the upper portion of the intake structure would increase slightly to accommodate the additional dedicated NMWC manifold piping. The bridge for the Joint SRWRS-ABFSHIP Elverta Diversion Alternative would need to be approximately 10 feet wider, approximately 40 feet total in width, to accommodate the new dedicated 72-inch-diameter pipe for NMWC. The additional width required for the bridge would make the intake structure a total of 250 feet long. The footprint area of the bridge support piers would increase proportionately.

3.7.1.4. Intake Support Facilities

Additional power would be required for the additional dedicated NMWC pumps but this would not change the approximate footprint of required facilities. In addition, backup power would not be provided for the NMWC pumps.

3.7.1.4.1. Primary Power Requirements, Availability, and Reliability

The maximum power requirement for the 371 mgd Intake Structure and the NMWC requirements have been estimated to be 5,450 kVA. **Table 3-6** generally summarizes power requirements. Power routing and other requirements would be similar to those outlined for the 235 mgd facility.

Table 3-6 Power Requirement Summary for 371 mgd Facility

Intake Pump Station	Peak Flow (mgd)	Pump Load[1] (hp)	Misc. Loads (kVA)	Power (kVA)	Amps @ 4,160 Volts	1/2 Load (kVA)
Intake Facility with Natomas Mutual Water Company	371	5,200	250	5,450	756	2,725

Notes:
[1] Includes a spare pump.
Key:
hp – horsepower kVA – kilovolt-ampere mgd – million gallons per day

3.7.1.4.2. Alternative Backup Power Supply Option

Since agricultural water operations typically have greater flexibility and are not required for emergency services, backup power would be provided only for the municipal portion of the intake. The required power facilities would be as previously described in **Section 3.5**.

3.7.2. Elkhorn Main Canal Modifications

As noted in **Chapter 1**, the capacity of the Elkhorn diversion is the CALFED-supported ABFSHIP Sankey/Elkhorn diversion alternative would be consolidated with the SRWRS Elverta Diversion Alternative to form the Joint SRWRS-ABFSHIP Elverta Diversion Alternative (refer to **Figure 3-21**). Based on information provided by NMWC and its engineering consultant, Mead and Hunt, the canal modifications portion of the ABFSHIP Sankey/Elkhorn diversion alternative currently includes upgrading and realigning approximately 1.6 miles of the NMWC's Elkhorn Main Canal. This reach of the canal runs from the Elkhorn Reservoir in the south, where the Elkhorn diversion with a capacity of 120 cfs was to be located, to the Central Main Canal in the north. The existing Elkhorn Main Canal is a 15- to 20-foot-wide earthen trough, 4 to 6 feet deep. The modified portions of the earthen canal would be uniformly trapezoidal in shape, with a 15- to 20-foot bottom approximately 5 feet deep, with 2:1 side slopes, and would be up to 40 feet wide at the top. The canal improvements would allow up to 150 cfs of water to be directed northward from the intake near the Elkhorn Reservoir to the Central Main Canal. Under the ABFSHIP Sankey/Elkhorn diversion alternative, the canal would also be relocated up to 30 feet to the east to reduce levee seepage concerns of Reclamation District 1000, the agency that maintains the Sacramento River levee in this reach. The canal modification work described in the ABFSHIP Sankey/Elkhorn diversion alternative also includes demolition, relocation, and modification of existing turnouts, drain sump pumps, concrete headwalls and culverts, piping, and utilities.

The planned canal modification portion of the Sankey/Elkhorn diversion alternative would be modified under the Joint SRWRS-ABFSHIP Elverta Diversion Alternative. The Elkhorn Main Canal would be required to slope both north and south from the Elverta Intake facility, rather than sloping only northward from the Elkhorn diversion, as previously planned. Additional check structures or turnout modifications may also be required to accommodate changes in water surface elevation due to this canal slope modification. An energy dissipation structure would be required to direct water into the canal from the Elverta Intake, as discussed in **Section 3.7.1.1** above. Additional modifications would also be required at the canal entrance structure at the Elkhorn Reservoir to provide a means for maintaining appropriate water surface elevations for wildlife habitat. The Joint SRWRS-ABFSHIP Elverta Diversion Alternative, similar to the SRWRS Elverta Diversion Alternative, would require the realignment of the Elkhorn Main Canal a maximum of 35 additional feet eastward beyond the planned realignment to accommodate the required levee raise and associated realignment of Garden Highway.

3.7.3. Construction and Operation of Facilities

Construction activities for the Elverta Intake would be similar to those described for the SRWRS Elverta Diversion Alternative. The extents of construction would be slightly larger due to the larger footprint of the joint intake. The width of the cofferdam would not change but the length would increase approximately 50 feet to create an overall length of 270 feet.

Extensive regrading along the entire existing approximately 1.6-mile Elkhorn Main Canal would be required. It is assumed that the canal would be constructed such that excavated material would be used to form the canal berms. More detailed analysis would be required to verify the cut and fill balance. Construction activities for the Elkhorn Main canal would include soil excavation, backfilling, and compaction. Excavated material would be used to form canal berms in areas where the canal would be realigned and/or widened. Small concrete structures containing weirs or gates would be constructed to control the water level at various locations in the canal. Drainage would be collected and piped to existing drainage ditches, or recirculated into the canal. Utilities would be relocated as required. Construction areas would be accessed directly off Elverta Road, and the staging area for the work would be coordinated with the staging area used for the proposed Elverta Intake Structure. Construction would be scheduled to avoid impacts on NMWC irrigation deliveries. Construction activities related to intake improvements and canal widening are anticipated to occur concurrently with construction of the other facilities.

NMWC would own the Elkhorn Main Canal and manage all operation and maintenance (O&M) activities, including control of hydrologic and hydraulic regimes during seasonal operations. Sacramento would own and operate the Elverta Intake facility.

3.7.4. North Natomas Water Treatment Plant

The North Natomas WTP location, facilities, construction activities and schedule, and O&M for this alternative would be the same as those described for the SRWRS Elverta Diversion Alternative.

3.7.5. Raw and Treated Water Pipelines

Raw and treated water pipeline routes, materials, support structures, construction activities and schedules, and O&M for this alternative would be the same as those described for the SRWRS Elverta Diversion Alternative.

3.7.6. Decommissioning of Elkhorn Pump Station

Under this alternative, NMWC would discontinue use of the Elkhorn pump station and the pump station would be decommissioned. Decommissioning would be performed in accordance with the standards of The Reclamation Board. Discharge pipes through the levee would be removed or abandoned in place by filling with concrete. The outfall, rubble and debris, and pumps would be removed. Wooden pilings in the river would be removed or cut off at the base. The historic pump house and pumping plant would be left. Pipes would be removed, along with walkways, for river pump platform access. Revegetation would be performed in accordance with permit conditions.

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CHAPTER 4 RAW WATER PIPELINES

The raw water pipeline conveys water from the intake facility to the WTP. This chapter describes the hydraulics (design flow, velocity, head loss, and pipe size) and the alignment the pipeline would follow, and provides characteristics about the pipeline and its construction that would be important for environmental documents. All elevations presented in this chapter are referenced to NGVD 29.

4.1. HYDRAULICS

The peak flow in the raw water pipeline would be 235 mgd. Two pipelines would convey the flow to provide redundancy should one pipe require maintenance, and also to make it possible to maintain higher velocities during low flow periods by using only one of the pipes. Pipeline length would be between 1 and 4 miles depending on the location selected for the WTP site. A WTP site located approximately 2.6 miles from the intake was assumed for illustration purposes. The head loss that would occur was calculated for pipelines of varying diameters. It was found that head loss, and therefore power cost, would increase more than pipe cost decreased if the two pipes were each any smaller than 78 inches in diameter. Two parallel pipes 78 inches in diameter each are recommended for the raw water pipeline. At peak flow, the velocity in these pipes would be approximately 5.5 fps. The head loss between the intake and the WTP would be between 4.5 and 20.5 feet, depending on the location selected for the WTP site. (Alternative WTP sites currently under consideration are described in **Chapter 5**; see **Figure 5-1**.) Typically, water elevations in the river vary from a low water level of elevation 4.28 to a high water elevation of 19.98 (the 10 percent exceedence value). The water would discharge into a grit removal chamber at the WTP. The exact elevation of this grit removal chamber would not be known until the plant design is complete. For planning purposes, it is estimated that the water elevation in the grit removal chamber would be 45 at a site located near the western end of potential WTP sites and 40 at a site located at the eastern end of potential WTP sites. Combining the lift and the head losses gives the range of total heads that would have to be pumped. Pumping heads would be highest when pipeline flows are highest and the river level is low, and heads would be lowest when pipeline flows are low and the river level is high. Predicted pumping heads for the range of conditions are given in **Table 4-1**.

Table 4-1 Range of Pumping Heads from Intake to Water Treatment Plant

Pipeline Discharge Point	Total Dynamic Pumping Head per Condition (feet)	
	Low River Level, Peak Intake Flow	High River Level, Low Intake Flow
To extreme western Water Treatment Plant site	45.2	25
To Water Treatment Plant site approximately 2.6 miles from intake assumed for illustrative purposes	53.2	25
To extreme eastern Water Treatment Plant site	56.2	20

4.2. ALIGNMENT SELECTED AND DESCRIPTION

From the intake, the raw water pipeline would follow an alignment as close to the projected toe of the existing levee as permitted by Reclamation District 1000. For planning purposes, the pipelines are estimated to be a minimum of 60 feet from the projected toe of the existing levee. The pipelines would continue south to Elverta Road and then turn east and follow Elverta Road to the WTP. An alternative route crossing diagonally through the field between the intake and Elverta Road was rejected because it would have required the purchase of additional right-of-way and would have impacted more sensitive habitat. Along Elverta Road, the pipes would be placed in the westbound lane, as close to the right-of-way line as possible. In areas where pipe construction would impact sensitive habitats on the roadside, the pipe would jog into or across the roadway. An overview of the pipeline alignment is shown in **Figure 1-1**. A more detailed view of the pipeline alignment and profile is presented in F**igure 4-1**. As noted previously, a WTP site located approximately 2.6 miles from the intake was assumed for illustrative purposes.

4.3. PIPE MATERIAL

Several materials would be suitable for this pipeline. The most common pipe types for this function and size are welded steel, ductile iron, and pretensioned concrete cylinder pipe. Final project specifications would be written for one or more of these three pipe types.

Should the pipe be steel, it would be coated and lined. The lining is usually cement mortar, although epoxy linings are occasionally used. The coating could be cement mortar, epoxy, or polyethylene tape. Cathodic protection may be used to protect the pipe from corrosion, depending on the corrosiveness of local soils. This would be determined during predesign investigations.

Should the pipe be ductile iron, it would have a cement mortar lining. The pipe would not have bonded coating, but would have polyethylene sleeves over the pipe for corrosion protection. Cathodic protection might be used, as with steel pipe.

No additional lining or coating is used with pretensioned concrete cylinder pipe. Cathodic protection may be used, as with steel pipe.

4.4. PIPELINE APPURTENANCES

The piping system would include valves at strategic locations. The intake pipes would have no branches; therefore, valves would only be used to isolate reaches of the pipe for maintenance. Isolation valves would be installed approximately every 1,000 feet along the pipe. The system would also include an air release valve at each high point and a blowoff at each low point. The air release valve assembly would be housed in a small aboveground enclosure located along the side of the road. The blowoff assembly would be entirely below ground. The system would also include access ports into the pipeline at intervals of approximately 1,000 feet.

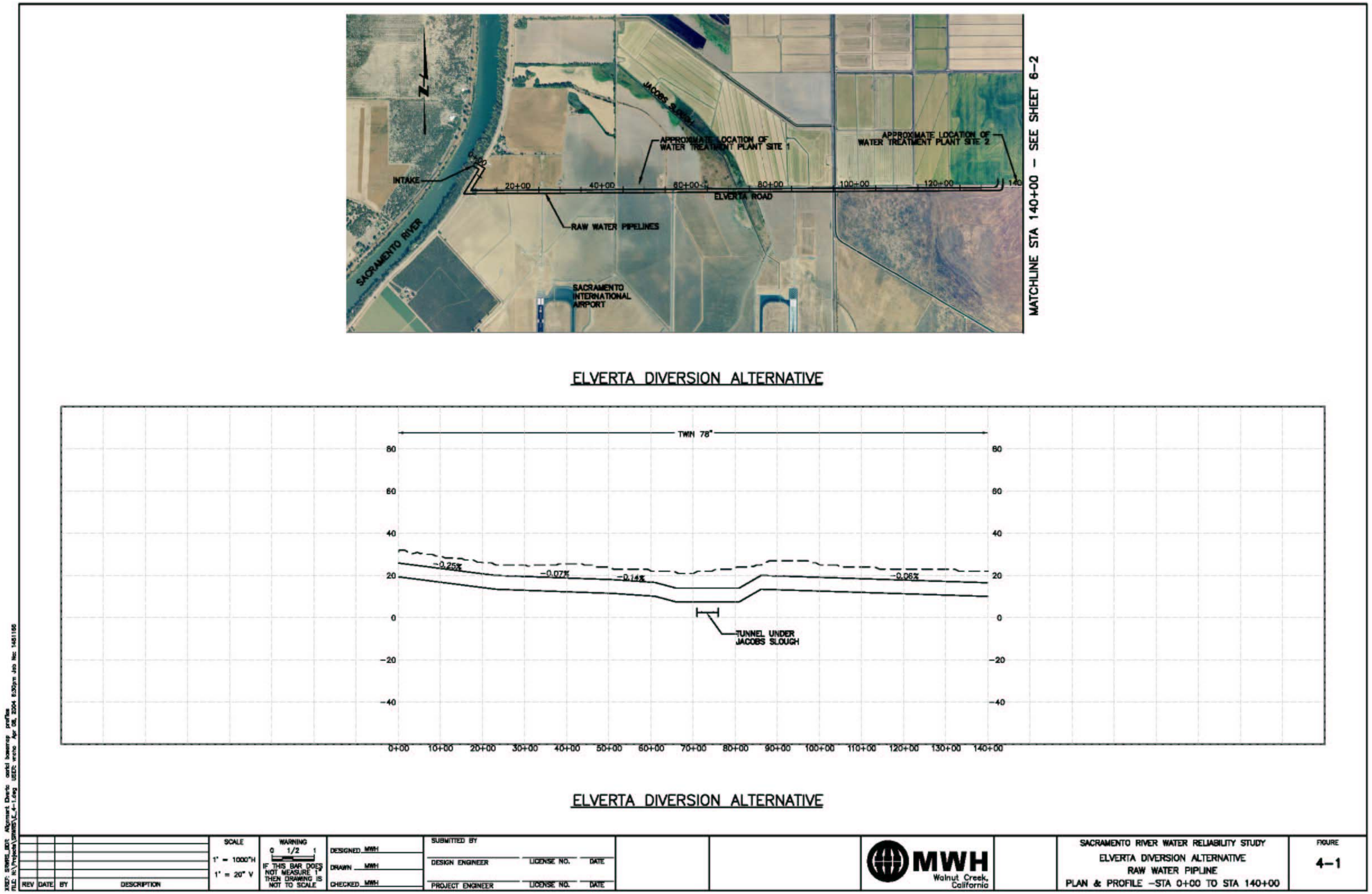

Figure 4-1 SRWRS Elverta Diversion Alternative Raw Water Pipeline Plan & Profile – STA 0+00 to STA 140+00

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4.5. CONSTRUCTION CHARACTERISTICS

The pipe trench would be typically 18 to 20 feet wide and 10 to 15 feet deep. Shoring would be used to maintain a narrow vertical side-wall trench and to protect workers. **Figure 4-2** presents a typical trench cross section. A work area at least 5 feet wide on one side of the trench and at least 15 feet wide on the other side of the trench would be needed for construction. Where available, a larger work area of up to 40 feet on one side of the trench would be provided to facilitate construction and reduce cost. Some of the work area could be achieved through temporary lane closures during work hours.

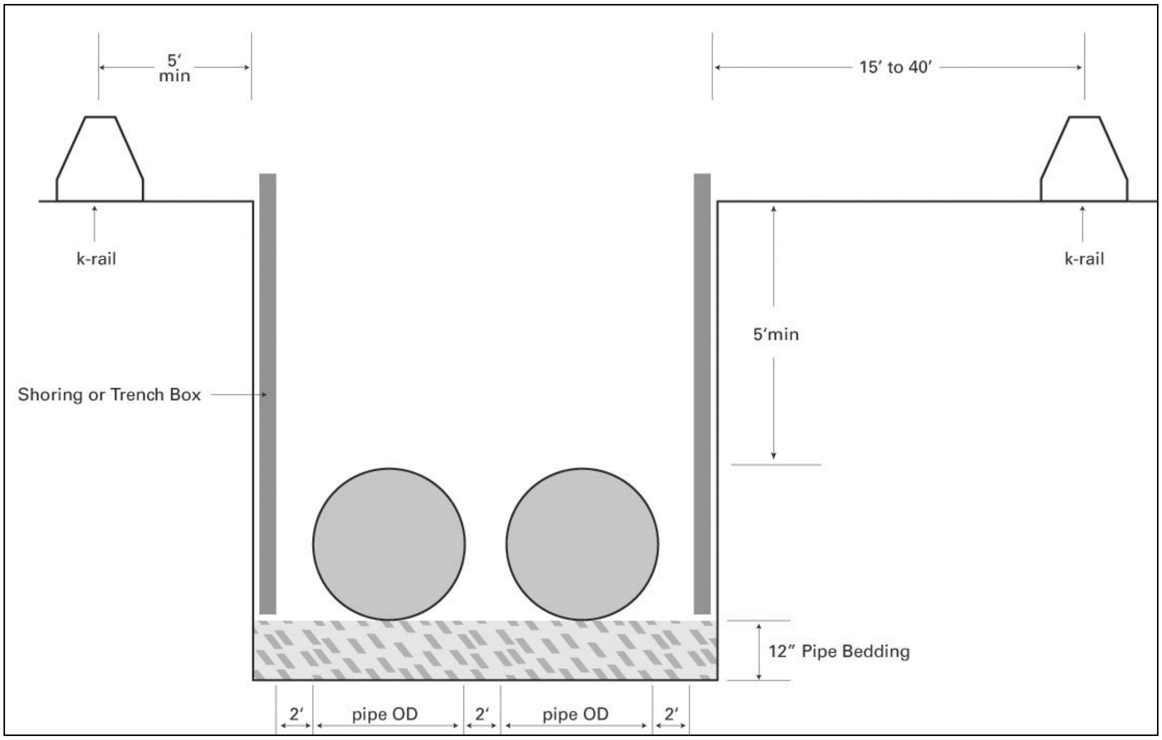

Figure 4-2 Typical Trench Section for Pipe Installation

Groundwater is high year-round in this area; therefore, extensive dewatering would be needed during construction, from before the trench is opened until after the trench is backfilled. Water removed from the construction area would be treated to remove sediment, and would be discharged to the closest drainage way. A discharge permit would be needed. The dewatering method most likely to be used is a network of well points along the pipeline alignment. The wells would be drilled to several feet below the trench invert, which would be 10 to 12 feet below grade. Well spacing could vary widely. Commonly, wells would be about 100 feet apart.

Pipe bedding would be crushed rock or sand. Pipe zone backfill would be sand or crushed rock or controlled density fill (very low strength concrete). Trench zone backfill would be native material. Any native materials unsuitable for trench backfill would be hauled away to a disposal site selected by the project sponsors.

Crews should be able to install pipe of this size and depth at production rates of 100 feet of trench per day during dry weather if no problems occur. However, to account for possible delays, average production rates would probably be about 40 feet of trench per day. **Table 4-2** presents estimated pipe lengths and construction durations for the raw water pipeline to a range of WTP sites. A contract period 40 work days longer than the construction period would be needed to allow for mobilization, demobilization, and punchlist work. Typical work days would be from 7:00 a.m. to 3:30 p.m. Monday through Friday, with work occasionally continuing as late as 7:00 p.m. and/or on Saturday.

Table 4-2 Estimated Construction Duration for the Raw Water Pipeline

Pipeline Discharge Point	Pipe Length (feet)	Trench Length (feet)	Construction Duration (work days)	Contract Period (work days)	Contract Period (calendar days)
To Water Treatment Plant site at extreme western End of possible sites	9,400	4,700	120	160	260
To Water Treatment Plant site at 2.6 miles from Intake	27,400	13,700	340	380	600
To Water Treatment Plant site at extreme eastern end of possible sites	47,000	23,500	590	630	1,000

The pipeline construction operation could use a number of different combinations of equipment. One possible scenario would include one or two excavators to excavate the trench, place pipe bedding and pipe zone backfill, and set the pipe; a front-end loader to move soil around the work site and load trucks; a dozer or tractor to move trench backfill into place; a large compactor and smaller walk-behind compactors; two to six end dump trucks to haul soil to and from the work site; and miscellaneous trucks to deliver materials and imported fill. Crew size would be 6 to 10 people, not including truck drivers. The crew superintendent and the contractor's project manager and field engineer may be local staff or, if the contractor is not a local contractor, may be brought in from outside the local area.

The number of truck trips to and from the construction site each day would vary depending on how much of the native soil can be used for backfill. If all the backfill can be native material taken from the trench and stored at the work area, only about 11 truck trips would be needed to haul away excess material, and 11 more truck trips to haul in imported material on an average day. Should the native material be unsuitable for backfill, or inadequate space exist at the work site to store the material until the trench is ready for backfill, the number of truck trips would increase to about 23 trips each to bring in material and haul away material.

Trucks hauling materials to and from the construction site would have loads with weights below highway load limits. Trucks hauling soil, rock, or sand to and from the job site would haul from 5 to 10 cubic yards of material in each load. Loads for other trucks would vary depending on what is being hauled, but would always be below H-20 load limits.

Safety on the construction site would be the responsibility of the construction contractor. The construction contractor would have a company safety program and a job-specific safety program administered by a project safety officer. Typical procedures would include weekly safety meetings with the construction crew and hazard analyses prepared before the beginning of each new operation. A traffic control plan would be prepared by the construction contractor and reviewed by Sacramento County to ensure traffic is safely routed around the work site. OSHA and Cal-OSHA standards would apply for all work.

No particularly noisy equipment would be anticipated for the construction work (e.g., no pile driving). Typical noise would include trucks and diesel-powered equipment. The work would comply with all county noise ordinances.

The construction contractor would have a staging area for field offices and to temporarily park equipment and supplies. This area would be 1 to 5 acres in size. A site has not been selected yet for this staging area. A 2- to 10-acre site also would be used for disposing excess material removed from the trench. Some material would be stockpiled only temporarily at the disposal site and then used later for backfill. Other material would be permanently placed at the disposal site. A grading permit would be obtained for the disposal site. Work at the disposal site would comply with all county requirements, including grading ordinance and sedimentation and erosion control requirements. A site has not yet been selected for this disposal site.

The raw water pipeline crosses one stream, Jacobs Slough, requiring a stream alteration permit from CDFG. It is not likely that the permit would allow using open-cut trenching to install the pipe across the stream; instead, tunneling would be used. A pressure balance tunneling technology would be used because the tunnel would be below groundwater levels. Tunneling would involve a jacking pit approximately 15 feet wide by 30 feet long by 25 feet deep on one side of the stream and a smaller receiving pit on the other side.

The construction contract documents would include a general SWPPP. The construction contractor would be required to submit a specific, more detailed SWPPP. The general plan would outline minimum requirements that must be met to minimize erosion and control sediments. The general and specific SWPPPs would comply with county sediment and erosion control ordinances. Typical best management practices that would be used include the following:

- Covering all exposed slopes and stockpiles with plastic, straw, or hydroseed
- Placing silt fences at the downstream side of all work areas
- Placing a sediment filter in each drop inlet
- Sweeping all work areas frequently
- Constructing sediment ponds in key locations
- Placing waddles or hay bales across steep, disrupted slopes
- Constructing gravel driveways at each work site exit
- Placing waddles or straw bales around the open trench work area

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CHAPTER 5 NORTH NATOMAS WATER TREATMENT PLANT

This chapter presents refined engineering for the water treatment facilities for the SRWRS Elverta Diversion Alternative. The North Natomas WTP would be designed for a maximum capacity of 235 mgd. Sacramento would be provided a peak flow of 145 mgd, which would serve as both a peaking supply and a base supply with an operating range between 20 and 145 mgd. PCWA would be provided a peak flow of 65 mgd, which could serve as a base supply and would serve as a peaking supply for the summer months with an operating range of 0 to 65 mgd. SSWD would be provided a peak flow of 15 mgd, which would serve as a base supply. Roseville would be provided a peak flow of 10 mgd, which may serve as a base supply with the potential use for aquifer storage and recovery (ASR) operations and peaking for summer months, with an operating range of 0 to 10 mgd.

The WTP would be located in the north Sacramento County area near Elverta Road, where the major transmission pipelines would be, but a final site has not yet been selected. The potential WTP location area is shown in **Figure 5-1**. The WTP would require a 90- to 100-acre site within the area shown. Sites located in the western portion of the potential WTP area would be in or near the approach to the two existing runways for Sacramento International Airport.

5.1. TREATED WATER GOALS AND OBJECTIVES

When planning water treatment facilities, it is necessary to identify goals and objectives for the treated water to guide in process selection, design of facilities, and development of an operations plan. The following are general goals and objectives for the treated water:

1. Treated water shall be potable and at a minimum meet all Federal and State drinking water standards.
2. Treated water shall be aesthetically pleasing to the consumer.
3. Treated water shall be provided to each SRWRS partner to blend with individual systems without creating distribution system water quality problems.
4. Treated water shall be provided reliably and as cost-effectively as possible.
5. Treated water shall have a sufficient disinfectant residual to provide delivery with detectable residual concentrations to SRWRS partners.
6. Treated water shall be non-corrosive to the SRWRS partners' distribution systems.

In addition to the above general goals and objectives, several more specific criteria have been set that directly impact design and operation of the water treatment facilities, including the following:

1. Water treatment facilities shall be designed to achieve appropriate microbial treatment, including a minimum of 3-log reduction of *Giardia*, 4-log reduction of viruses, and 2-log reduction of *Cryptosporidium* (subject to water quality monitoring data) through physical removal and chemical inactivation.
2. Filters shall be designed for filter-to-waste operation after a backwash.
3. Filters shall be designed with an auxiliary backwash system, using either air scour or surface wash.

4. Combined filter effluent turbidity shall be less than 0.1 nephelometric turbidity unit (NTU) at all times.
5. Individual filter effluent turbidity shall be less than 0.1 NTU within 1 hour of bringing the filter online or after a backwash until the end of the filter run.
6. Facilities shall be designed for recycle of waste washwater decant. This recycle shall occur as necessary and be limited to less than 10 percent of WTP flow. All recycle streams shall be equalized prior to return. Sludge decant shall be managed through an alternative management strategy, such as disposal to the sewer or treatment and discharge.

5.2. REGULATORY BACKGROUND

This section reviews current and anticipated drinking water regulations as promulgated by the United States Environmental Protection Agency (USEPA) and the California Department of Health Services (DHS). Under the provisions of the Safe Drinking Water Act (SDWA), DHS has the primary enforcement responsibility (referred to as “primacy”). The Health and Safety Code of the California Administrative Code establishes DHS authority and stipulates drinking water quality and monitoring standards. To maintain primacy, a State’s drinking water regulations can be no less stringent than the Federal standards (a State’s regulations can be more stringent).

USEPA and DHS establish primary regulations for controlling contaminants that affect public health, and secondary regulations for compounds that affect the taste or aesthetics of drinking water. For each contaminant that is regulated, USEPA is required to establish a maximum contaminant level (MCL) or a treatment technique (TT) to limit the level of these compounds in drinking waters. USEPA is also required to recommend a Best Available Technology (BAT) for removing each contaminant during treatment.

Figure 5-1 Elverta Diversion Alternative Potential WTP Sites

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5.2.1. Current Regulations

The most significant drinking water quality regulations are shown in **Table 5-1**. **Appendix A** summarizes each contaminant in drinking water currently regulated by both USEPA and DHS. The table identifies the regulation and the MCL or the TT associated with each contaminant listed. The following is a general discussion of the requirements of selected regulations.

Table 5-1 Summary of Major Federal and State Drinking Water Quality Regulations

Regulation	Year of Promulgation	Number of Contaminants	Targeted Contaminants
National Interim Primary Drinking Water Regulations (NIPDWR)	1975-1981	7	Trihalomethanes, Arsenic, Radiologicals
Phase I Standards	1987	8	VOCs
Phase II Standards	1991	36	VOCs, SOCs, and IOCs
Phase V Standards	1992	23	VOCs, SOCs, and IOCs
Surface Water Treatment Rule (SWTR)	1989	5	Microbiological and Turbidity
Total Coliform Rule (TCR)	1989	2	Microbiological
Lead and Copper Rule (LCR)	1991/2003 (1)	2	Lead and Copper
Drinking Water Source Assessment and Protection Program	1996	-	Source Water Protection
Information Collection Rule (ICR)	1996	-	Microbiological and D/DBPs
Stage 1 Disinfectants/Disinfection By-Products (D/DBP) Rule	1998	14	D/DBPs and Precursors
Stage 2 D/DBP Rule	2006	9	DBPs
Interim Enhanced Surface Water Treatment Rule (ESWTR)	1998	2	Microbiological and Turbidity, Systems >10,000 people
Long-Term 2 ESWTR	2006	1	*Cryptosporidium*
Unregulated Contaminant Monitoring Rule	1999	36	Organics and Microbiological
Radionuclides Rule	2000	4	Radionuclides
Arsenic Rule	2001	1	Arsenic
Filter Backwash Rule	2001	-	Microbiological and Turbidity
Long-Term 1 ESWTR	2002	2	Microbiological and Turbidity, Systems <10,000 people
Drinking Water Candidate Contaminant List	2003	9	Various

Note:
(1) California Adoption of Federal Rule Minor Revisions.
Key:
D/DBP – disinfectants/disinfection by-products
ESWTR – Enhanced Surface Water Treatment Rule
ICR – Information Collection Rule
IOC – inorganic compounds
NIPDWR – National Interim Primary Drinking Water Regulations
VOC – volatile organic compound
SOC – synthetic organic compound
SWTR – Surface Water Treatment Rule
TCR – Total Coliform Rule

5.2.1.1. Surface Water Treatment Rule

The Surface Water Treatment Rule (SWTR) was promulgated to control the levels of turbidity, *Giardia lamblia*, viruses, *Legionella*, and heterotrophic plate count bacteria in United States drinking waters. Many of the detailed requirements of this regulation would be enhanced or superceded by the Interim and Long-Term 2 Enhanced Surface Water Treatment Rules (LT2ESWTR), described later.

The California SWTR requires all utilities using a surface water supply or a groundwater supply under the influence of a surface water supply, to provide adequate disinfection, and under most conditions, to provide filtration. Exemptions from filtration of surface water supplies are provided in rare occasions when the source water supply meets extremely rigid requirements for water quality and the utility possesses control of the watershed.

5.2.1.1.1. <u>General Requirements</u>

The SWTR includes the following general requirements to minimize human exposure to microbial contaminants in drinking water:

- Utilities are required to achieve at least 99.9 percent removal and/or inactivation of *Giardia lamblia* cysts (3-log removal) and a minimum 99.99 percent removal and/or inactivation of viruses (4-log removal). The required level of removal/inactivation must occur between the point where the raw water ceases to be influenced by surface water runoff to the point at which the first customer is served.
- The disinfectant residual entering the distribution system must not fall below 0.2 milligrams per liter (mg/L) for more than 4 hours during any 24-hour period.
- A disinfectant residual must be detectable in 95 percent of distribution system samples. A heterotrophic plate count (HPC) concentration of less than 500 colonies/milliliter (mL) can serve as a detectable residual if no residual is measured.
- Each utility must perform a watershed sanitary survey at least every 5 years.

5.2.1.1.2. <u>Removal Credit</u>

The level of removal credit given a utility for both *Giardia lamblia* and viruses is determined by the type of treatment process used. For a conventional WTP, the SWTR provides a 2.5-log removal credit for *Giardia lamblia* and a 2.0-log removal credit for viruses.

5.2.1.1.3. <u>Disinfection Credit</u>

Disinfection during conventional treatment (assuming all operational criteria and performance standards are met, and the plant receives 2.5-log credit for physical removal of *Giardia* and 2-log credit for physical removal of viruses) must achieve 0.5-log inactivation of *Giardia lamblia* and 2.0-log inactivation of viruses. To determine the inactivation of *Giardia lamblia* and viruses achieved at a WTP, the SWTR established the concept of CT. CT is the product of the concentration of disinfectant remaining at the end of a treatment process ("C" in mg/L) and the contact time in which 10 percent of the water passes through the treatment process ("T" or "T10" in minutes). The contact time in which 10 percent of the water travels through a unit process can be conservatively estimated from DHS guidelines or more accurately determined by conducting a tracer study. The USEPA Guidance Manual for the SWTR includes tables that identify the log removal of both *Giardia lamblia* and viruses achieved for a calculated CT value based on the type of disinfectant, the water temperature, and pH.

5.2.1.2. Stage 1 Disinfectants and Disinfection By-Products Rule

The purpose of the Stage 1 Disinfectants/Disinfection By-Product (D/DBP) Rule is "...to minimize risks from disinfection by-products and still maintain adequate control over microbial contamination."

5.2.1.2.1. Maximum Residual Disinfectant Level Goals

The USEPA has set maximum residual disinfectant level goals (MRDLG) for chlorine, chloramines, and chlorine dioxide, as shown in **Table 5-2**.

Table 5-2 Maximum Residual Disinfectant Level Goals

Disinfectant	Goal
Chlorine	4 mg/L as Cl_2
Chloramines	4 mg/L as Cl_2
Chlorine Dioxide	0.8 mg/L as ClO_2

Key:
mg/L – milligrams per liter

MRDLGs are set at levels for which no known or anticipated adverse health effects occur. These goals are non-enforceable health goals based only on health effects and exposure information.

5.2.1.2.2. Maximum Residual Disinfectant Levels

The Stage 1 D/DBP Rule established maximum residual disinfectant levels (MRDL) for chlorine, chloramines, and chlorine dioxide, as shown in **Table 5-3**.

Table 5-3 Maximum Residual Disinfectant Levels

Disinfectant	Level
Chlorine	4.0 mg/L as Cl_2
Chloramines	4.0 mg/L as Cl_2
Chlorine Dioxide	0.8 mg/L as ClO_2

Key:
mg/L – milligrams per liter

The residual disinfectant level must be monitored at the same points in the distribution system and at the same time as when sampling for total coliforms. Compliance with the MRDL would be based on the running annual average of the monthly average of all samples, computed quarterly. Plant operators could increase the residual disinfectant level in the distribution system above the MRDL if necessary to protect public health from acute microbiological contamination problems, including distribution line breaks, storm runoff events, source water contamination, or cross-connections.

5.2.1.2.3. Maximum Contaminant Level Goals for TTHMs, HAA5, Chlorite, and Bromate

The USEPA has set maximum contaminant level goals (MCLG) for four trihalomethanes, two haloacetic acids, chlorite, and bromate, as shown in **Table 5-4**. (The MCLG for chloroform was removed by the USEPA on May 30, 2000.)

The MCLGs are set at levels for which no known or anticipated adverse health effects occur. These goals are non-enforceable health goals based only on health effects and exposure information.

Table 5-4 Maximum Contaminant Level Goals

Disinfection By-Product	Goal
Bromodichloromethane	0 mg/L
Dibromochloromethane	0.06 mg/L
Bromoform	0 mg/L
Dichloroacetic Acid	0 mg/L
Trichloroacetic Acid	0.3 mg/L
Chlorite	0.8 mg/L
Bromate	0 mg/L

Key:
mg/L – milligrams per liter

5.2.1.2.4. Maximum Contaminant Levels for TTHMs, HAA5, Chlorite, and Bromate

The Stage 1 D/DBP Rule set MCLs for total trihalomethane (TTHM), five haloacetic acids (HAA5), chlorite, and bromate, as shown in **Table 5-5**.

Table 5-5 Maximum Contaminant Levels

Contaminant	Level
TTHM[(1)]	0.080 mg/L
HAA5[(2)]	0.060 mg/L
Chlorite	1.0 mg/L
Bromate	0.010 mg/L

Notes:
[(1)]TTHM includes chloroform, bromodichloromethane, dibromochloromethane, bromoform.
[(2)] HAA5 includes mono-, di- and tri-chloroacetic acids and mono- and di-bromoacetic acids.
Key:
mg/L – milligrams per liter

Total Trihalomethanes and Haloacetic Acids. TTHMs and HAA5 are formed when disinfectants react with naturally occurring organic matter in water. All systems must monitor the distribution system for TTHMs and HAA5. Compliance for surface water, groundwater under the direct influence of surface water (GWUDIS), and groundwater systems with a population greater than 10,000 is based on the running annual average of quarterly averages of all samples taken in the distribution system, computed quarterly.

5.2.1.2.5. Treatment Technique for Disinfection By-Product Precursors

The USEPA requires systems that have surface water or GWUDIS as a supply, and use conventional filtration treatment, to remove specific amounts of organic material by implementing a treatment technique, either by enhanced coagulation or enhanced softening. The percent of removal required depends on source water total organic carbon (TOC) and alkalinity. **Table 5-6** summarizes removal requirements.

Compliance with this treatment technique must be calculated on a quarterly basis after 12 months of data are available. Each month the system must calculate percent actual TOC removal, determine the percent required TOC removal (from above), and calculate the removal ratio (must be greater than 1.0).

Table 5-6 TOC Removal Requirements

TOC Level (mg/L)	Removal Percentage by Alkalinity Level		
	0 – 60 (mg/L)	> 60 – 120 (mg/L)	> 120 (mg/L)
> 2.0 - 4.0	35	25	15
> 4.0 - 8.0	45	35	25
> 8.0	50	40	30

Key:
mg/L – milligrams per liter TOC – total organic carbon

Systems can be granted a 1.0 ratio for the monthly removal ratio under the four following conditions (regardless of the calculated removal ratio):

- Remove greater than or equal to 10 mg/L of magnesium hardness (as calcium carbonate ($CaCO_3$))
- Raw water TOC is less than 2.0 mg/L
- Raw water or treated water specific ultraviolet absorbance (SUVA) is less than or equal to 2.0 liters per milligram-meter (L/mg-m)
- Treated water alkalinity is less than 60 mg/L (only for systems practicing enhanced softening)

The USEPA has also provided alternative compliance criteria from the treatment technique requirements. Utilities would not be required to achieve specified TOC removals provided one of the following conditions is met:

- Source water TOC is less than 2.0 mg/L
- Treated water TOC is less than 2.0 mg/L
- Source water TOC is less than 4.0 mg/L, source water alkalinity is greater than 60 mg/L, and distribution system TTHM is less than 0.04 mg/L and HAA5 is less than 0.03 mg/L
- Distribution system TTHM is less than 0.04 mg/L and HAA5 is less than 0.03 mg/L, and only chlorine is used for primary disinfection and distribution system residual
- Source water SUVA, prior to any treatment, is less than or equal to 2.0 L/mg-m
- Treated water SUVA is less than or equal to 2.0 L/mg-m

5.2.1.3. Stage 2 Disinfectants and Disinfection By-Products Rule

The Stage 2 D/DBP Rule was published in January 2006. It applies to public water systems, community water systems (CWS) or nontransient noncommunity water systems (NTNCWS), that add a primary or residual disinfectant other than ultraviolet light or deliver water treated with a primary or residual disinfectant other than ultraviolet light.

The key provision in this rule is the change in calculating the MCL. Currently, compliance with the MCL is calculated using a running annual average (RAA) to average compliance samples from all distribution system sampling locations. Under the Stage 2 D/DBP Rule, the MCL will be calculated using locational running annual averages (LRAA). PWSs must maintain the LRAA for each compliance sampling location at or below 0.080 mg/L TTHM and 0.060 mg/L HAA5. All systems, including consecutive systems, must comply with the MCLs for TTHM and HAA5 LRAA using compliance sampling locations identified from the Initial Distribution System Evaluation (IDSE) Final Report.

5.2.1.3.1. Initial Distribution System Evaluation

An IDSE will be performed to identify locations with representative high TTHM and HAA5 concentrations throughout a system's retail distribution system. The IDSE results will be used in conjunction with the Stage 1 D/DBP Rule compliance monitoring to identify and select Stage 2 D/DBP Rule routine compliance monitoring locations. There are four IDSE options:

- Standard monitoring program
- System specific study (based on TTHM and HAA5 monitoring) and modeling requirements
- Obtainment of a 40/30 waiver
- Obtainment of a very small system waiver

Both the timing and number of IDSEs and routine compliance monitoring are based on the retail population served by the individual public water system(s). The timing of when the IDSE must be completed is based on either an individual system's retail population, or in the case of a combined distribution system, the retail population served by the largest system in that combined system. The numbers of IDSE samples in the standard monitoring option are based on each individual system's retail population.

5.2.1.3.2. Compliance Monitoring

Compliance with the Stage 2 D/DBP Rule will be based on calculating a LRAA, where compliance means maintaining the annual average at each compliance sampling location in the distribution system at or below 0.080 mg/L and 0.060 mg/L for TTHM and HAA5, respectively. This is in lieu of the RAA MCL calculation under the Stage 1 D/DBP Rule that averaged observed values across distribution system compliance sampling locations. Monitoring for the LRAA will occur at compliance sampling locations identified in the IDSE Final Report at specific frequencies based on system population.

If a water system is required to conduct quarterly monitoring, it must make compliance calculations at the end of the fourth calendar quarter that follows the compliance date and at the end of each subsequent quarter (or earlier if the LRAA calculated based on fewer than four quarters of data would cause the MCL to be exceeded regardless of the monitoring results of subsequent quarters). If a system is required to conduct monitoring at a frequency that is less than quarterly, it must make compliance calculations beginning with the first compliance sample taken after the compliance date.

5.2.1.3.3. Operational Evaluation Levels

The Stage 2 D/DBP Rule includes the concept of operational evaluation levels. Operational evaluation levels trigger a system to evaluate system operational practices and identify opportunities to reduce DBP concentrations in the distribution system to reduce the potential the system will exceed the MCL. The

Stage 2 DBP operational evaluation levels are identified using the system's Stage 2 D/DBP Rule compliance monitoring results.

Operational evaluation levels are calculated as follows:

If (Q1 + Q2 +2Q3)/4 > MCL, then the system must conduct an operational evaluation

Where:

Q3 = current quarter measurement
Q2 = previous quarter measurement
Q1 = quarter before previous quarter measurement
MCL = Stage 2 MCL for TTHM (0.080 mg/l) or Stage 2 MCL for HAA5 (0.060 mg/L)

The operational evaluation includes an examination of system treatment and distribution operational practices, including changes in sources or source water quality, storage tank operations, and excess storage capacity, which may contribute to high TTHM and HAA5 formation. Systems must also identify steps that could be considered to minimize future operational evaluation level exceedences.

5.2.1.3.4. Minimum Reporting Levels for Disinfection By-Products

The rule establishes regulatory minimum reporting limits (MRL) for compliance reporting of DBPs by public water systems. These regulatory MRLs also define the minimum concentrations that must be reported as part of the Consumer Confidence Reports. Beginning April 1, 2007, quantitative data must be reported for concentrations at least as low as those listed for all DBP samples analyzed for compliance.

5.2.1.3.5. Maintain TOC < 4 mg/L for Reduced TTHM and HAA5 Monitoring

To qualify for reduced routine compliance monitoring for TTHM and HAA5, subpart H systems (i.e., systems that use surface water supplies or GWUDIS) that are not monitoring to demonstrate compliance with TOC removal requirements of Stage 1 D/DBP Rule (i.e., plants that are not conventional filtration designs) must take TOC samples every 30 days at a location prior to any treatment, beginning April 1, 2008 or earlier, if specified by the State. The source water TOC RAA must be <4.0 mg/L (based on the most recent four quarters of monitoring) on a continuing basis at each treatment plant to reduce or remain on reduced monitoring for TTHM and HAA5. After demonstration of the TOC level compliance, the system may reduce monitoring to every 90 days.

Systems on a reduced monitoring schedule may remain on that reduced schedule as long as the average of all samples taken in the year (for systems that must monitor quarterly), or the results of the sample (for systems that must monitor no more frequently than annually) are no more than 0.060 mg/L and 0.045 mg/L for TTHMs and HAA5, respectively.

5.2.1.4. Interim Enhanced Surface Water Treatment Rule

The Interim Enhanced Surface Water Treatment Rule (ESWTR) applies to public water systems that use surface water or GWUDIS and serve a population greater than 10,000. The purpose of this regulation is "…to improve control of microbial pathogens, including specifically *Cryptosporidium*, in drinking water; and address risk trade-offs with disinfection by-products."

5.2.1.4.1. <u>Cryptosporidium</u>

The Interim ESWTR set an MCLG of zero (0) for the protozoan genus *Cryptosporidium*. Since no reliable means exists for monitoring this constituent in the drinking water at the time of promulgation, a treatment technique requirement was established in lieu of setting an MCLG. The treatment technique requires a 2-log (99 percent) *Cryptosporidium* removal or control for public water systems that are currently required to filter under the existing SWTR. This removal must be achieved between the raw water intake and the first customer.

The rule provides that systems with conventional or direct filtration WTPs would be granted the 2-log removal credit if turbidity requirements are met for the existing SWTR (1.0/5.0 NTU) and the combined filter effluent requirements for this rule (0.3/1.0 NTU).

The rule also provides that systems with slow sand or diatomaceous earth filtration WTPS would be granted the 2-log removal credit if turbidity requirements are met for the existing SWTR (1.0/5.0 NTUs).

5.2.1.4.2. <u>Turbidity</u>

For surface water and GWUDIS systems that are required to filter their source water under the existing SWTR, and that employ conventional or direct filtration for treatment, the combined filter effluent turbidity requirements have been tightened. For alternative filtration technologies, the State would set turbidity performance requirements at a level that, in combination with disinfection, would consistently achieve 99.9 percent removal/inactivation of *Giardia*, 99.99 percent removal/inactivation of viruses, and 99 percent removal of *Cryptosporidium*.

The combined filter effluent turbidity must be less than 0.3 NTU in 95 percent of measurements and may never exceed 1 NTU (based on 4-hour measurements). The combined filter effluent turbidity shall not exceed 1.0 NTU for more than 8 hours (based on 15-minute measurements). Combined filter effluent and individual filter effluent continuous turbidity monitoring shall be recorded every 15 minutes. Monthly reports must show the total number of measurements taken, and have two options for value reporting:

- Report 15-minute measurements and show the 50th, 90th, 95th, 98th, and 99th percentiles, and report all measurements greater than 1 NTU
- Report 4-hour measurements and show all results greater than 0.3 NTUs (based on 15-minute measurements), and percent of measurements less than or equal to 0.3 NTUs (based on 15-minute measurements).

The rule requires continuous, online measurement of turbidity for each individual filter. These data must be recorded every 15 minutes. Systems with two or fewer filters may conduct continuous monitoring of the combined filter effluent turbidity in lieu of individual monitoring. Individual filter effluent turbidity monitoring shall be less than 0.3 NTUs within 60 minutes after return to service.

DHS is expected to add several other requirements to the rule, including the following:

- All filters shall be visually inspected once per year as part of the operations plan based on DHS guidance.
- Raw water shall be sampled for total coliform and either fecal coliform or *E. Coli* at least once per month.
- Chlorine residual shall be confirmed in 95 percent of distribution samples every month.

- Online turbidimeters shall be manually verified once per week for combined filter effluent and once per month for individual filter effluent.
- Turbidity shall be recorded and reported for sedimentation effluent at least once per day.
- Flow rate and turbidity shall be recorded and reported for recycled backwash water at least once per day.
- System must report turbidity data to the State within 10 days after the end of each month.

5.2.1.4.3. Disinfection Profiling and Benchmarking

The purpose of disinfection profiling and benchmarking is to develop a process to assure no significant reduction in microbial protection occurs as a result of significant disinfection process modifications to meet the new MCLs for TTHMs and HAA5 from the Stage 1 D/DBP Rule.

Profiling would be required for surface water systems that have either TTHM levels greater than or equal to 80 percent of the new MCL (0.064 mg/L) or HAA5 levels greater than or equal to 80 percent of the new MCL (0.048 mg/L).

The disinfection profile is developed using a minimum of 1 year of weekly *Giardia lamblia* log inactivation. The month with the lowest average log inactivation shall be identified as the critical period or benchmark.

After profiling and benchmarking is complete, a utility must submit this information to the State as part of the sanitary survey. If a utility decides to make changes to disinfection practices, the utility must consult with the State to ensure that microbial protection is not compromised.

5.2.1.4.4. Finished Water Reservoirs

Under this rule, surface water and GWUDIS systems must cover all new treated water reservoirs, holding tanks, and other storage facilities.

5.2.1.4.5. Sanitary Surveys

Primacy states, such as California, must now conduct sanitary surveys for all surface water and GWUDIS systems, regardless of size. These surveys must be conducted every 3 years CWSs and every 5 years for noncommunity water systems. DHS may grant a waiver to water utilities to perform the sanitary survey every 5 years if the system has outstanding performance based on previous sanitary surveys. DHS must determine how outstanding performance would be evaluated to allow for the reduced frequency of the sanitary survey.

Sanitary surveys must meet the eight components of the 1995 USEPA/State Guidance. These components include source assessment, treatment, distribution system, finished water storage, pumps, pumping facilities and controls, monitoring and reporting, data verification, system management and operation, operator compliance with state requirements, and disinfection profiling (if required).

5.2.1.5. Long-Term 2 Enhanced Surface Water Treatment Rule

The LT2ESWTR was published by USEPA in early January 2006 in the Federal Register. This regulation will apply to all public water systems that use surface water or GWUDIs.

The LT2ESWTR includes deadlines that directly affect drinking water utilities of all sizes, and many will have to meet deadlines later this year. Some systems serving more than 100,000 people will have to submit detailed monitoring plans under the LT2ESWTR by July 1, 2006. The Major Milestone Schedule for Stage 2 D/DBP Rule and LT2ESWTR Implementation provides an overview of key monitoring, reporting, and compliance milestones under both rules.

The requirements for filtered and unfiltered systems are different. This section summarizes only the requirements for filtered systems.

5.2.1.5.1. Source Water Monitoring

Filtered systems are not required to conduct source water monitoring if the system will provide a total of at least 5.5-log of treatment for *Cryptosporidium*. Otherwise, PWSs using surface water or GWUDI are required to monitor their source water (i.e., the influent water entering the treatment plant) monthly for 24 months to determine an average *Cryptosporidium* level. As described in the next section, monitoring results determine the extent of *Cryptosporidium* action requirements under the LT2ESWTR. Large systems must also monitor for *E. coli* and turbidity at the same time in source water.

Systems must adhere to the sampling plan and report results no later than 10 days after the end of the first month following the month when the sample is collected. All systems serving at least 10,000 people must report the results from the initial source water monitoring to USEPA electronically using the Central Data Exchange (CDX). Submission of historical (grandfathered) data is allowed when it meets the quality assurance and quality control requirements specified in the rule.

Systems serving less than 10,000 persons may use *E. coli* as a surrogate indicator for *Cryptosporidium*. However, if the *E. coli* levels are sufficiently high, these systems must then undertake *Cryptosporidium* monitoring.

The rule also includes a second round of *Cryptosporidium* sampling for all systems. This second round of sampling will take place 6 years following bin classification for the source water.

5.2.1.5.2. Analytical Method

Systems must analyze for *Cryptosporidium* using either USEPA Method 1623 or Method 1622. Systems must analyze at least a 10-liter (L) sample or a packed pellet volume of at least 2 mL. The rule contains specific quality assurance and quality control requirements. Only EPA-approved laboratories can perform the *Cryptosporidium* sample analysis. Specific analytical methods are also specified for turbidity and *E. coli* measurements required by the rule.

5.2.1.5.3. Sampling

Filtered systems serving at least 10,000 people must sample their source water for *Cryptosporidium*, *E. coli*, and turbidity at least monthly for 24 months. Filtered systems serving fewer than 10,000 people must sample their source water for *E. coli* at least once every 2 weeks for 12 months. Filtered systems serving fewer than 10,000 people must sample their source water for *Cryptosporidium* at least twice per month for 12 months or at least monthly for 24 months if the system does not conduct *E. coli* monitoring, or if the initial E. coli sample exceed the following criteria:

- For systems using lake/reservoir sources, the annual mean *E. coli* concentration is greater than 10 *E. coli*/100 mL.
- For systems using flowing stream sources, the annual mean *E. coli* concentration is greater than 50 *E. coli*/100 mL.

Systems must collect samples within a 5-day period around the schedule date. If an extreme condition or situation exists that may pose danger to the sample collector, or that cannot be avoided and causes the system to be unable to sample, the system must sample as close to the scheduled date as is feasible unless the State approves an alternative sampling date. The system must submit an explanation for the delayed sampling date to the State concurrent with the shipment of the sample to the laboratory. If a system is unable to report a valid analytical result for a scheduled sampling date due to equipment failure, loss of or damage to the sample, failure to comply with the analytical method requirements, including the quality control requirements, or the failure of an approved laboratory to analyze the sample, then the system must collect a replacement sample.

Replacement samples should be collected not later than 21 days after receiving information that an analytical result cannot be reported for the scheduled date unless the system demonstrates that collecting a replacement sample within this time frame is not feasible or the State approves an alternative re-sampling date. The system must submit an explanation for the delayed sampling date to the State concurrent with the shipment of the sample to the laboratory. Systems that fail to meet these criteria for any source water sample must revise their sampling schedules to add dates for collecting all missed samples. Systems must submit the revised schedule to the state for approval prior to when the system begins collecting the missed samples.

5.2.1.5.4. Monitoring Location

Systems must collect samples for each plant that treats a surface water or GWUDI source. Where multiple plants draw water from the same influent, such as the same pipe or intake, the State may approve one set of monitoring results to be used for all plants. Systems must collect source water samples prior to chemical treatment, such as coagulants, oxidants, and disinfectants. The State may approve a system to collect a source water sample after chemical treatment. To grant this approval, the State must determine that collecting a sample prior to chemical treatment is not feasible for the system and that the chemical treatment is unlikely to have a significant adverse effect on the analysis of the sample. Systems that recycle filter backwash water must collect source water samples prior to the point of filter backwash water addition. Specific requirements are included from bank filtration and other special cases.

A system that begins using a new source of surface water or GWUDI after the system is required to begin monitoring under paragraph (c), the monitoring section of the LT2ESWTR, and must monitor the new source on a schedule the State approves.

5.2.1.5.5. Monitoring and Treatment Compliance Dates

Starting dates for monitoring are staggered by system size, with smaller systems beginning monitoring after larger systems. Milestones for monitoring, reporting, and compliance occur first for very large systems (>100,000 persons), then systems serving 50,000 to 99,999 persons, followed by systems serving 10,000 to 49,999 persons, and finally systems serving fewer than 10,000. Populations are based on retail population.

5.2.1.5.6. Bin Classification Table for Filtered Systems

Filtered water systems will be classified in one of four categories or bins based on their monitoring results. The rule specifies several calculation procedures depending on how many samples were collected or if the sample frequency was not consistent.

Bin Placement may be calculated as follows:

- Total of at least 48 samples; the bin concentration is equal to the arithmetic mean of all sample concentrations.
- Total of at least 24 samples, but not more than 47 samples; the bin concentration is equal to the highest arithmetic mean of all sample concentrations in any 12 consecutive months during which *Cryptosporidium* samples were collected.
- For systems that serve fewer than 10,000 people and monitor for *Cryptosporidium* for only one year (i.e., collect 24 samples in 12 months), the bin concentration is equal to the arithmetic mean of all sample concentrations.
- For systems with plants operating only part of the year that monitor fewer than 12 months per year under § 141.701(e) of the LT2ESWTR, the bin concentration is equal to the highest arithmetic mean of all sample concentrations during any year of *Cryptosporidium* monitoring.

Additional action for *Cryptosporidium* (beyond 3.0-log reduction awarded for conventional filtration) will be based on source water concentrations of the protozoa and the type of treatment implemented at the plant. If the maximum running annual average (MRAA) is less than 0.075 oocysts/L, the source is assigned Bin 1 classification and no additional action is required. Assuming conventional filtration credit, if the MRAA is between 0.075 and 1.0 oocysts/L, the source is assigned to Bin 2 and 1-log action is required; if the MRAA is between 1.0 and 3.0 oocysts/L, the source is assigned to Bin 3 and 2-log action required; and if the MRAA is greater than 3.0 oocysts/L, the source is assigned to Bin 4 and 2.5-log action required.

Systems classified in Bins 2, 3, and 4 must provide 1.0- to 2.5-log additional action for *Cryptosporidium*. Systems will select from a wide range of treatment and management strategies in the "microbial toolbox" to meet their additional action requirements. Systems classified in Bin 3 and Bin 4 must achieve at least 1 log of additional treatment using either one or a combination of the following: bag filters, bank filtration, cartridge filters, chlorine dioxide, membranes, ozone, or ultraviolet (UV) light.

5.2.1.5.7. Microbial Toolbox

PWSs can achieve additional *Cryptosporidium* treatment credit through implementing pretreatment processes, such as presedimentation or bank filtration, by developing a watershed control program, and by applying additional treatment steps like ozone, chlorine dioxide, UV, and membranes. In addition, PWSs can receive a higher level of credit for existing treatment processes through achieving superior filter effluent turbidity or through a demonstration of performance. Taken as a whole, this list of control options is termed the "microbial toolbox." PWSs may use one or more tools to accumulate the needed treatment credits to meet the treatment requirement associated with their bin classification.

5.2.1.5.8. UV Dose Table

Systems receive *Cryptosporidium*, *Giardia lamblia*, and virus treatment credits for UV light reactors by achieving the UV dose values described in the rules. Systems must validate and monitor UV reactors to demonstrate that they are achieving a particular UV dose value for treatment credit. UV reactor validation must occur at full-scale using a test microbe with quantified dose-response characteristics using low-pressure mercury lamps. Validation must include operating conditions of flow rate, UV intensity as measured by a UV sensor, and UV lamp status, as well as other considerations, including lamp fouling and inlet/outlet hydraulics. To receive treatment credit for UV light, systems must treat at least 95 percent of the water delivered to the public during each month by UV reactors operating within validated conditions for the required UV dose.

5.2.1.5.9. CT Tables

CT is the product of the disinfectant contact time (T, in minutes) and disinfectant concentration (C, in mg/L). Systems with treatment credit for chlorine dioxide or ozone must calculate CT at least once each day, with both C and T measured during peak hourly flow. Systems with several disinfection segments in sequence may calculate and sum the CT for each segment, where a disinfection segment is defined as a treatment unit process with a measurable disinfectant residual level and a liquid volume. Systems receive the *Cryptosporidium* treatment credit by meeting the corresponding CT value for the applicable water temperature specified in CT tables specified in the rule.

5.2.1.5.10. Open Finished Water Reservoirs

Until now, regulations required PWSs to cover all new storage facilities for finished water but did not address existing uncovered finished water storage facilities. Under the LT2ESWTR, PWS using uncovered finished water storage facilities must either cover the storage facility or treat the storage facility discharge to achieve inactivation and/or removal of 4-log virus, 3-log *Giardia lamblia*, and 2-log *Cryptosporidium* on a State-approved schedule.

5.2.1.5.11. Microbial Profiling and Benchmarking

Following the completion of initial source water monitoring (date varies by system size), a system that plans to make a significant change to its disinfection practice must develop disinfection profiles and calculate disinfection benchmarks for *Giardia lamblia* and viruses. Significant changes to disinfection practice are defined as follows:

- Changes to the point of disinfection
- Changes to the disinfectant(s) used in the treatment plant
- Changes to the disinfection process
- Any other modification identified by the State as a significant change to disinfection practice

5.2.1.6. Arsenic Rule

The Final Arsenic Rule was promulgated by the USEPA on January 22, 2001. The rule sets an MCLG of 0 mg/L and an MCL of 0.010 mg/L (10 micrograms per liter (µg/L)) for arsenic. DHS has not yet adopted this regulation and the State version may be more stringent (see later discussion).

5.2.1.7. Filter Backwash Recycling Rule

The Final Filter Backwash Recycling Rule applies to all PWSs that use surface water and employ conventional or direct filtration and recycle water within the WTP.

This rule requires all recycle streams to pass through all treatment processes; therefore, all streams need to be returned prior to chemical addition and coagulation. Also, each system must notify DHS in writing that it practices recycling. This notification must include a plant schematic that shows the type and location of recycle streams, typical recycle flow data, highest plant flow in the previous year, design flow of the plant, and DHS-approved operating capacity.

Each system must collect and maintain the following information: copy of recycle notice to DHS, list of all recycle flows and frequency, average and maximum backwash flow rate and duration, typical filter run length and how determined, type of recycle treatment, and data on recycle treatment facilities.

5.2.2. Anticipated Regulations

The USEPA and DHS are developing new regulations. Major anticipated regulations that would impact surface water supplies are shown in **Table 5-7**, and selected regulations are discussed below.

Table 5-7 Summary of Anticipated Major Federal and State Drinking Water Quality Regulations for Surface Water Supplies

Regulation	Year Final Expected	Number of Contaminants	Targeted Contaminants
Perchlorate (1)	2004	1	Perchlorate
Arsenic (2)	2004	1	Arsenic
Hexavalent Chromium(1)	2004	1	Hexavalent Chromium
Drinking Water Candidate Contaminant List/ Unregulated Contaminant Monitoring Rule	2007	-	Microbiological and Chemical
Distribution System Rule/Revised Total Coliform Rule	2008	-	Microbiological

Notes:
(1) California rule only.
(2) California adoption of Federal rule expected to be more stringent.

5.2.2.1. California Arsenic Regulation

DHS is required to develop a revised arsenic standard for drinking water in California by June 30, 2004. This may be delayed due to change in the governor's administration. The Office of Environmental Health Hazard Assessment (OEHHA) has developed a Public Health Goal (PHG) for arsenic of 4 nanograms per liter (ng/L). This is well below the current MCL of 10 µg/L. DHS is currently developing a revised MCL using this information.

5.2.2.2. California Hexavalent Chromium Regulation

DHS was required to develop a new hexavalent chromium standard for drinking water in California by January 1, 2004. This has been delayed due to change in the governor's administration. OEHHA repealed the PHG of 0.2 µg/L and OEHHA was to final a PHG in 2003. DHS plans to develop an MCL for hexavalent chromium shortly after publication of the PHG.

5.2.2.3. Drinking Water Candidate Contaminant List/ Unregulated Contaminant Monitoring Rule

The 1996 SDWA Amendments provided a list of chemical and microbial contaminants for possible future regulation. Every 5 years, USEPA selects at least five contaminants from the list and determines whether to continue to regulate them. The regulations would be determined based on risk assessment and cost-benefit considerations and on minimizing overall risk. USEPA developed a draft second list for determination in April 2004.

The Unregulated Contaminant Monitoring Rule (UCMR) requires CWSs to conduct “treated” water monitoring of specified unregulated constituents. The purpose is to assist USEPA in collecting information about contaminants present in drinking water supplies that are currently unregulated. In agreement with the Contaminant Candidate List, the next UCMR, expected in 2004 or 2005, would be revised to reflect current constituents of concern.

5.2.2.4. Distribution System Rule/Revised Total Coliform Rule

USEPA conducted a review of 69 existing drinking water regulations in April 2002. USEPA determined only the Total Coliform Rule (TCR) was a candidate for revision. USEPA conducted two meetings with experts to identify major distribution system issues. From these meetings, nine white papers were developed on the most critical subjects, including the following:

- Cross connection control
- Aging infrastructure and corrosion
- Permeation and leaching
- Nitrification
- Biofilms/growths
- Covered storage
- Decay in water quality over time
- New/repaired water mains

USEPA plans to publish a revised TCR by 2006 and a final rule by 2008.

5.3. WATER QUALITY EVALUATION

Below is a summary of water quality related to the Elverta Intake site on the Sacramento River. Several monitoring programs were queried to obtain available water quality data between 1992 and 2002.

Table 5-8 summarizes those programs and the data acquired.

Table 5-8 Monitoring Program Summary

Program	Monitoring Period	Parameters	Location of Sample Site(s)
USGS National Ambient Water Quality Assessment Program	February 1996 through April 1998	• Total dissolved solids • Dissolved organic carbon • General water quality: iron, temperature, conductivity, pH, alkalinity, hardness, suspended solids • Rice herbicides: molinate, thiobencarb, carbofuran	• Feather River near Nicolaus • Sacramento River at Verona
Sacramento River Watershed Program	June 1998 through May 2002	• Total dissolved solids • Coliforms, protozoa • Dissolved organic carbon at UV254 • General water quality: nutrients, metals, minerals, temperature, conductivity, pH, alkalinity, hardness, suspended solids, turbidity • Organics: diazinon, molinate, thiobencarb	• Feather River at Nicolaus • Sacramento River at Veteran's Bridge
SRCSD Coordinated Monitoring Program	December 1992 through June 2002	• Total dissolved solids • Coliforms, protozoa • Total and dissolved organic carbon at UV254 • General water quality: nutrients, metals, minerals, temperature, conductivity, pH, alkalinity, hardness, suspended solids, turbidity • Organics	• Sacramento River at Veteran's Bridge
DWR Municipal Water Quality Investigation	April 1994 through February 1998	• Total dissolved solids • Trihalomethane formation potential • Dissolved bromide	• Sacramento River at Bryte Bend Water Treatment Plant
City of West Sacramento	January 1995 through December 1999	• Total dissolved solids • Coliforms • UV254 • General Water Quality: nutrients, metals, minerals, temperature, conductivity, pH, alkalinity, hardness, turbidity	• Sacramento River at Bryte Bend Water Treatment Plant

Key:
DWR – California Department of Water Resources
SRCSD – Sacramento Regional County Sanitation District
USGS – United States Geological Survey
UV259 – Ultraviolet 254

5.3.1. Description of Sampling Locations

Several sampling locations are identified in the monitoring programs used as resources for water quality data. One monitoring site is located upstream at Verona, one just downstream at Veteran's Bridge, and one farther downstream at Bryte Bend. These sites have available data for developing a characterization of the water for treatment purposes. No major discharges into the Sacramento River exist between Verona and Bryte Bend, but in-river activities and minor discharges occur that would cause some deterioration of the water quality downstream. Below is a brief description of each sampling location, including its location relative to the Elverta Intake location.

- Sacramento River at Verona: This sample site is located on the Sacramento River, just downstream from the confluence with the Feather River. At this location, both major agricultural drains (Colusa Basin Drain and Sacramento Slough) have discharged to the Sacramento River. This site is located upstream from the Elverta Intake site.
- Sacramento River at Veteran's Bridge: This sample site is located on the Sacramento River at Veteran's Bridge, the Interstate 5 crossing, just downstream from the Elverta Intake site.
- Sacramento River at Bryte Bend WTP: This sample site is located on the Sacramento River, near the Interstate 80 crossing, located downstream from the Elverta Intake site.

5.3.2. Discussion of Water Quality

The Sacramento River has very good quality surface water. Downstream locations generally are more susceptible to contaminating activities and therefore may have a less preferable water quality. However, agricultural herbicide concentrations typically occur higher upstream. The source water can be treated to meet all existing State and Federal drinking water standards.

On the Sacramento River, a substantial agricultural input into the river occurs downstream from the confluence with the Feather River. The primary agricultural use is rice farming. Pesticides associated with rice farming, including molinate, thiobencarb, and carbofuran, have primary and secondary drinking water standards. Monitoring data are available at several locations, and all data show that both primary and drinking water standards can be met. The secondary standard for thiobencarb is approached closely (a maximum value of 0.7 µg/L, as compared with the MCL of 1.0 µg/L) at several monitoring locations during the spring months of application. This may be a taste concern in the distribution system if not pre-treated. The cities of West Sacramento and Sacramento have facilities in place at their existing WTPs to conduct oxidation with potassium permanganate if necessary.

Table 5-9 summarizes the available general water quality of the Sacramento River near the Elverta Intake site, specific to drinking water purposes.

Table 5-9 General Water Quality of the Sacramento River near Elverta Diversion Site

Constituent	Minimum	Maximum	Average	Median
Alkalinity, mg/L				
Verona	24	73	54	55
Veteran's Bridge	16	77	60	61
Bryte Bend WTP	25	92	58	58
Bromide, mg/L Bryte Bend WTP	<0.01	0.07	0.01	0.01
DOC, mg/L				
Verona	1.3	3.6	1.9	1.6
Veteran's Bridge	0.7	10	3.0	3.0
Hardness, mg/L				
Verona	24	69	52	54
Veteran's Bridge	28	97	59	59
Iron, mg/L				
Veteran's Bridge	0.356	2.0	0.86	0.61
Manganese, mg/L				
Veteran's Bridge	0.028	0.107	0.057	0.047
pH, units				
Verona	7.5	8.1	7.8	7.8
Veteran's Bridge	6.2	8.9	7.7	7.7
Bryte Bend WTP	6.7	8.4	7.6	7.6
Suspended Sediment, mg/L Verona	24	117	59	49
Specific Conductance, µS/cm				
Verona	62	186	131	135
Veteran's Bridge	21	316	155	155
Temperature, Celsius				
Verona	8.7	22.5	15.4	14.3
Veteran's Bridge	7.5	25	15.1	14.8
Bryte Bend WTP	7.2	25.2	16.4	16.1
TDS, mg/L				
Veteran's Bridge	11.5	165	105	104
Bryte Bend WTP	<50	135	89	88
TOC, mg/L Veteran's Bridge	<0.2	6.6	2.9	3.0
TSS, mg/L Veteran's Bridge	4	200	41	32
Turbidity, NTU				
Veteran's Bridge	3.75	81.2	26.6	24.6
Bryte Bend WTP	7	387	34	25
UV254, cm-1 Veteran's Bridge	0.0606	0.14	0.103	0.105

Key:
cm-1 – absorbance per centimeter
DOC – dissolved organic carbon
mg/L – milligrams per liter
µS/cm – microSiemens per centimeter
NTU – nephelometric turbidity unit
TOC – total organic carbon
TDS – total dissolved solids
TSS – total suspended solids
WTP – water treatment plant

Graphs of temperature, pH, total suspended solids (TSS), and organic carbon have been generated to look at the seasonal variability of each constituent (see **Figure 5-2** through **Figure 5-5**).

These graphs show that temperature is greatly affected by season, and is very predictable, with the lowest levels in the winter months (7 degrees Celsius (ºC)) and the highest levels in the late summer months (over 22 ºC). The average temperature is 15 ºC.

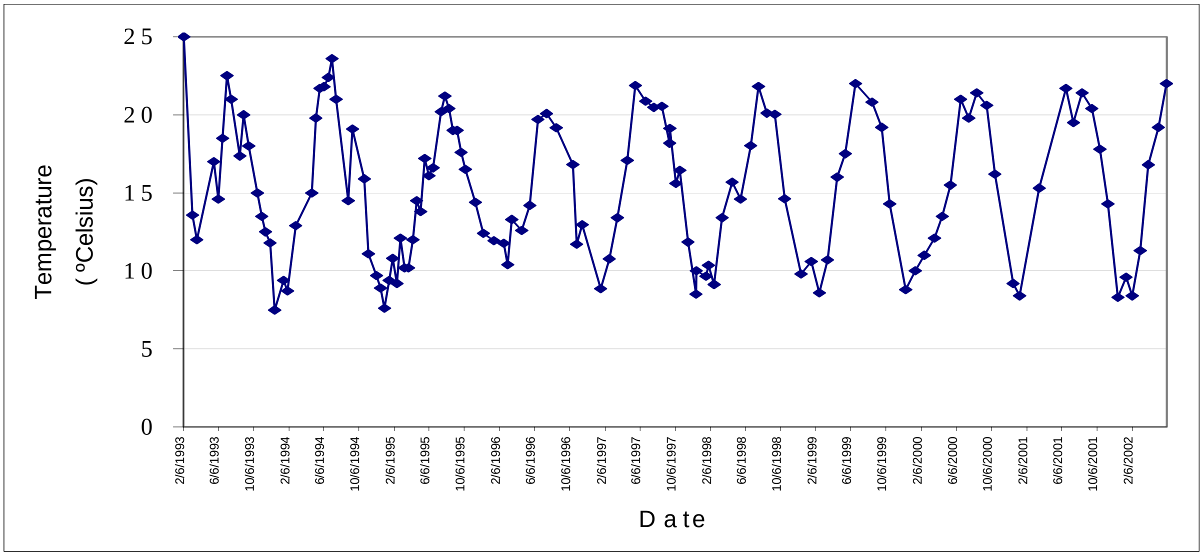

Figure 5-2 Temperature Levels in Sacramento River at Veteran's Bridge Coordinated Monitoring Program

Values for pH are also variable, but have less predictability than temperature. Extreme lows occur near 6.5 pH units and extreme highs near 9 pH units, but generally pH varies between 7 and 8 pH units.

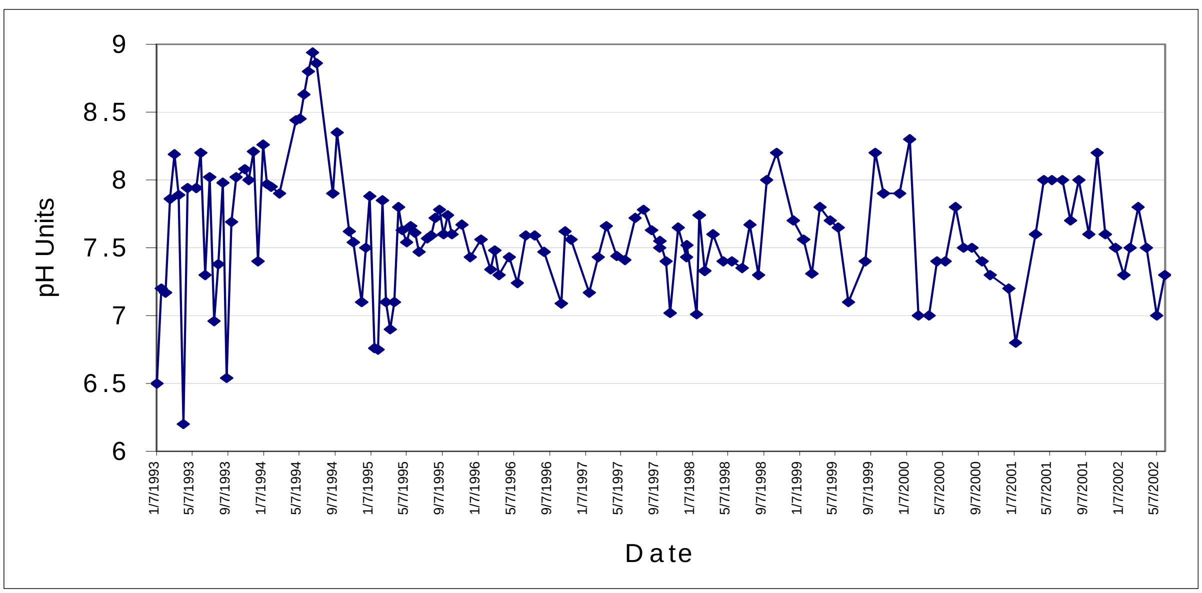

Figure 5-3 pH Levels in Sacramento River at Veteran's Bridge Coordinated Monitoring Program

TSS is highest during the winter months. This measurement is typically around 1.2 times higher than turbidity measurements. This shows that TSS can range from 4 to 200 mg/L, resulting in probable turbidity levels of 3 to 165 NTU, with an average of 28 to 34 NTU. This solids load is likely associated with wet weather events and releases from upstream reservoirs.

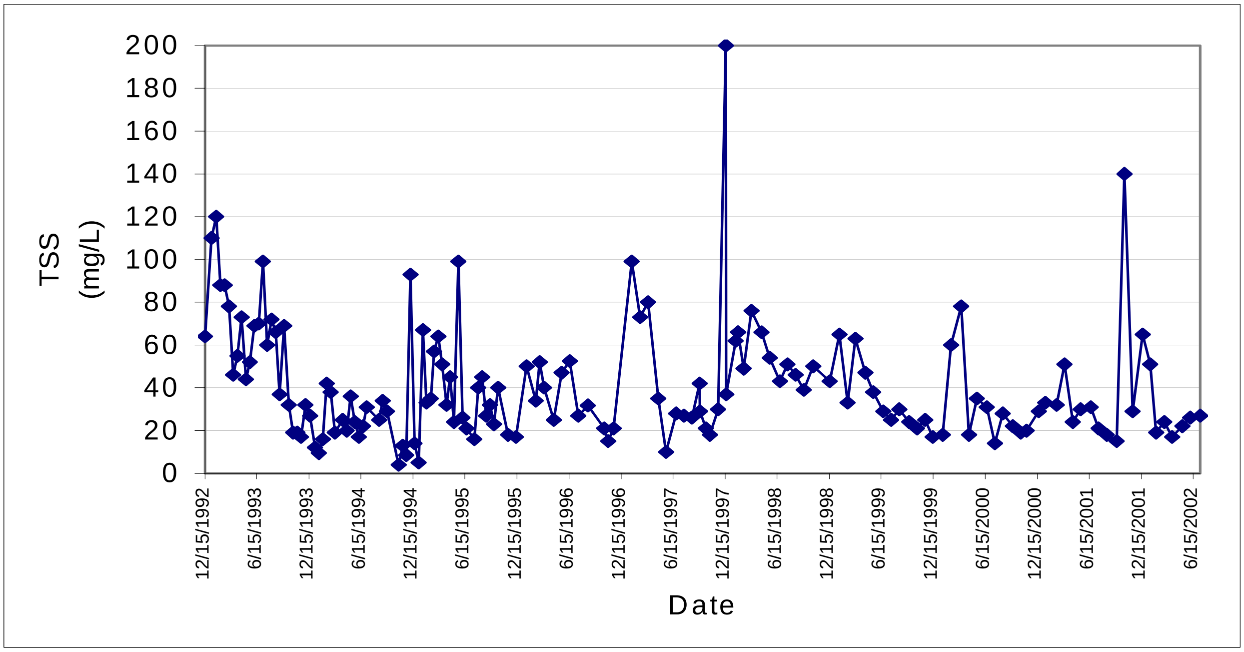

Figure 5-4 Total Suspended Solids in Sacramento River at Veteran's Bridge Coordinated Monitoring Program

TOC is a measure of the organic carbon in water and is recognized as a general indicator of the occurrence of DBP precursor material. The Coordinated Monitoring Program (CMP) monitors ambient river levels for TOC and dissolved organic carbon (DOC) (small particulate carbon). These data show that levels can range from 0.2 to 5.2 mg/L, with an average value just over 2 mg/L. The highest levels are seen in the late fall and winter months and the lowest levels are seen through the summer and early fall months. Intake data collected by the City of West Sacramento at the Bryte Bend WTP show only sporadic winter detects of TOC greater than 2 mg/L.

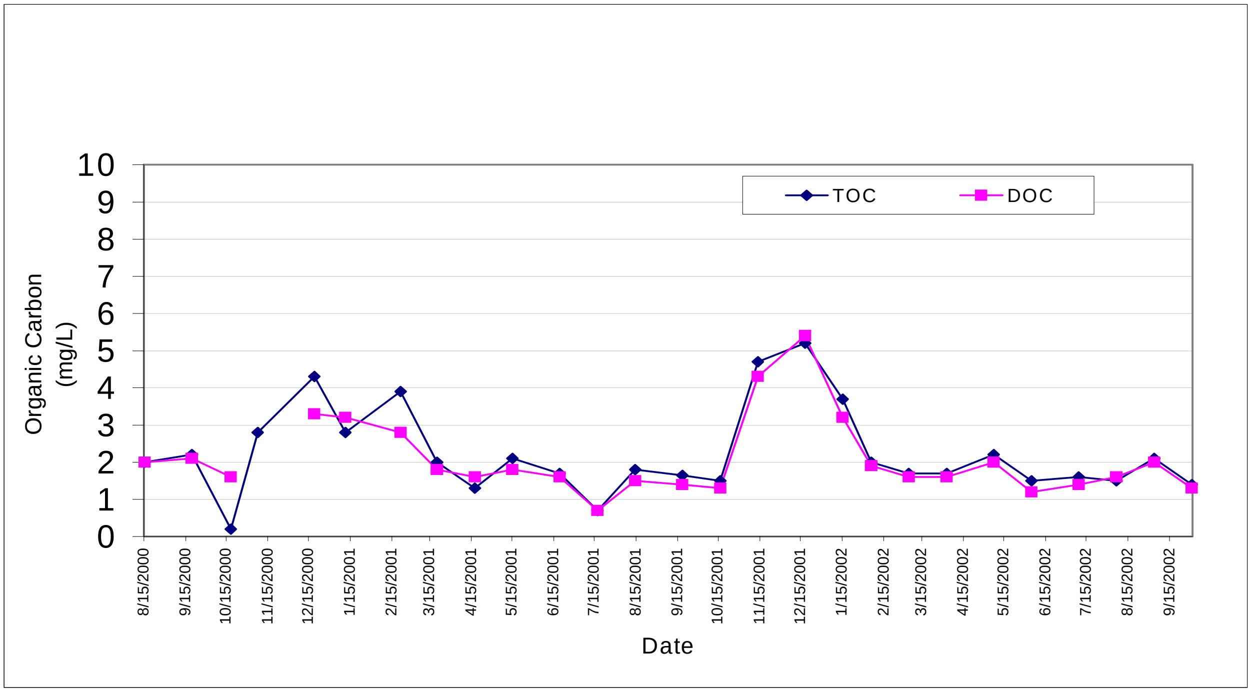

Figure 5-5 Organic Carbon Levels in Sacramento River at Veteran's Bridge Coordinated Monitoring Program

Data also are available for microbial constituents. Coliforms and protozoa have been monitored since 1998 at various sites. **Table 5-10** summarizes the available data.

Giardia and *Cryptosporidium* detects were primarily found in the late fall and early winter months. Average levels are low enough to ascertain that 3/4-log reduction of *Giardia* and viruses and 2-log reduction of *Cryptosporidium* are expected to be appropriate for the North Natomas WTP.

Table 5-10 Microbial Water Quality of the Sacramento River Near Elverta Intake

Constituent	Minimum	Maximum	Geometric Mean
Total Coliform, MPN/100 mL			
Veteran's Bridge	17	16,000	480
Bryte Bend WTP	<2	>16,000	460
Fecal Coliform, MPN/100 mL			
Veteran's Bridge	2	2400	30
Bryte Bend WTP	<2	1300	30
E. Coli, MPN/100 mL			
Veteran's Bridge	<2	300	20
Bryte Bend WTP	<2	3000	20
Constituent	**No. of Samples**	**No. of Samples Positive**	**12-Month Running Annual Average**
Giardia, cysts/L	38	6	0.058
Cryptosporidium, oocysts/L	38	2	0.033

Key:
L – liter mL – milliliter MPN/100 mL – most probable number per 100 milliliters WTP – water treatment plant

A graph of coliform over the sampling period is provided in **Figure 5-6**. It can be seen that coliform can peak throughout the year, but are mostly associated with wet weather events.

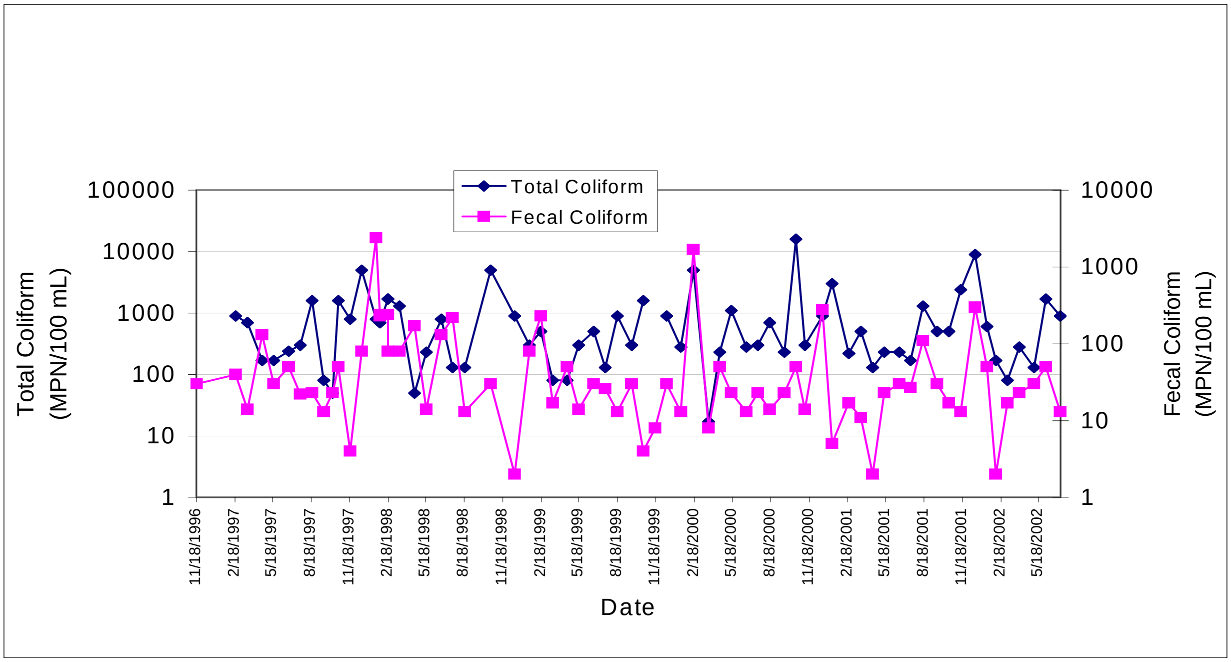

Figure 5-6 Coliform Levels in Sacramento River at Veteran's Bridge Coordinated Monitoring Program

5.3.3. Overview

Based on water quality data collected from other monitoring programs in the vicinity of the proposed Elverta Intake site, and presented herein, it appears that water quality is expected to be very good. It is recommended that a monitoring program be implemented at the proposed intake site to collect further data for turbidity, pH, alkalinity, TOC, *E. Coli*, *Giardia*, and *Cryptosporidium.* The SRWRS partners may also want to petition the California Department of Pesticide Regulation to monitor spring herbicide levels at the proposed diversion site during the 2006 or 2007 monitoring program to compare with the Bryte Bend and Sacramento River WTP intakes.

5.4. PROCESS IDENTIFICATION AND DESIGN CRITERIA

In this section, the water treatment process is identified, and design criteria are discussed for filtration facilities, solids handling facilities, chemical feed and supply systems, and electrical. In addition, sewer and stormwater management, site configuration and layout, special considerations, and construction characteristics are discussed.

5.4.1. Water Treatment Process Selection

Using the water quality data obtained and summarized above, 3/4/2-log reduction of *Giardia*/viruses/*Cryptosporidium* has been identified as the likely level of treatment required under the SWTR and LT2ESWTR. In addition to treating microbial constituents, removing rice herbicides may be desired seasonally. Therefore, oxidation facilities using potassium permanganate (KMnO4) would be provided. In the event that additional log reduction of *Cryptosporidium* is required in the future, both physical and hydraulic space would be reserved on site to allow for future installation of UV light facilities.

Conventional filtration with chlorine disinfection is proposed for the North Natomas WTP because it is employed widely, is reliable for treating water with seasonal variability in its quality, and because its long hydraulic detention time allows plant performance to be less sensitive to abrupt hydraulic or raw water quality changes. Long sedimentation detention times improve the removal of solids and TOC and assist in controlling taste and odors during treatment. Conventional filtration also involves high capital costs and a large facility footprint due to the need to construct large facilities.

The North Natomas WTP would be designed for a plant capacity flow rate of 235 mgd. **Table 5-11** summarizes the preliminary design values for each treatment process. A preliminary process flow diagram has been developed (shown in **Figure 5-7**, at end of chapter), which includes the following processes:

- Grit basin
- Flash mix
- Flocculation/sedimentation basin
- Dual media gravity filtration
- Future UV light
- Chlorine contact tank (Sacramento only – to achieve 0.5 log *Giardia* inactivation)
- Clearwell (for WTP operational storage only at 10 percent of plant capacity)

Roseville delivers fluoridated water to its consumers; therefore, a remote chemical feed facility would be required to fluoridate the treated water from the North Natomas WTP. Fluoridation typically depresses the pH; therefore, caustic soda would be needed to increase the pH to meet Roseville's distribution system requirements. Finally, sodium hypochlorite would be needed to ensure that an adequate disinfectant residual was maintained. Roseville has not yet identified a parcel for this facility; a parcel would be located during land use planning for this service area. A summary of the proposed facilities has been developed to present the likely components of this facility and is included in **Appendix B**.

Table 5-11 Preliminary Design Values for Water Treatment Processes

Description	Units	Preliminary Design Value
PLANT CAPACITY		
Design Flow	mgd	235
	cfs	365
	gpm	163,823
GRIT BASINS		
Type	-	Horizontal Flow
Grit Basin Flow Rate	mgd	235
	gpm	163,823
Number of Basins	no.	4
Width of Basins	ft	35
Length of Basins	ft	170
Water Depth	ft	12
Volume, total	cu ft	285,600
Detention Time	min	13.0
Surface Loading	gpm/sf	6.9
Grit Collection Type	-	Chain & Flight
FLASH MIX SYSTEM		
Type	-	Pump Diffusion
Number of Systems	no.	3
Mixing Energy	sec^{-1}	750 - 1,000

Table 5-11 Preliminary Design Values for Water Treatment Processes (cont.)

Description	Units	Preliminary Design Value
FLOCCULATION BASINS		
Type	-	Vertical Turbine
Number of Basins	no.	6
Basin Width	ft	80
Basin Length	ft	100
Number of Compartments per Basin	no.	16
Compartment Width	ft	19.5
Compartment Length	ft	19.5
Water Depth	ft	16
Volume, each basin	cu ft	128,000
Volume, total	cu ft	768,000
Flocculation Detention Time	min	26.7
Basin Detention Time	min	35.1
Number of Stages	no.	4
Flocculators per Stage	no.	4
Total Number of Flocculators	no.	96
Mixing Energies		
Stage 1	sec^{-1}	60/30
Stage 2	sec^{-1}	40/20
Stage 3	sec^{-1}	30/15
Stage 4	sec^{-1}	20/10
SEDIMENTATION BASINS		
Type	-	Rectangular, Horizontal Flow
Number of Basins	no.	6
Basin Width	ft	80
Basin Length	ft	370
Water Depth	ft	16
Width: Length Ratio	-	0.22
Volume, each	cu ft	473,600
Volume, total	cu ft	2,841,600
Detention Time	min	129.7
Surface Loading	gpm/sq ft	0.92
Sludge Removal Type	-	Chain & Flight

Table 5-11 Preliminary Design Values for Water Treatment Processes (cont.)

Description	Units	Preliminary Design Value
FILTERS		
Type	-	Constant Level, Constant Rate
Filter Flow Rate	mgd	235
	gpm	163,823
Number of Filters	no.	24
Bays per Filter	no.	2
Width of Bay	ft	18
Length of Bay	ft	38
Media Area per Filter	sq ft	1,368
Total Filter Media Area	sq ft	32,832
Filtration Rate		
All Filters in Service	gpm/sq ft	5.0
One Filter Not in Service	gpm/sq ft	5.2
Filter Media		
Anthracite Coal		
Depth	inch	30
Effective Size	mm	1.0
Sand		
Depth	inch	12
Effective Size	mm	0.5
Total Depth	inch	42
Total L/D Ratio	-	1,372
Gravel Depth	inch	None
Filter Backwash System		
Underdrain Type	Concrete Plenum with Nozzles	
Backwash Rate	gpm/sq ft	18
Backwash Duration	min	10
Filter Auxiliary Wash System		
Type	-	Air Scour
Wash Rate	scfm/sq ft	3.5
	min	3

Table 5-11 Preliminary Design Values for Water Treatment Processes (cont.)

Description	Units	Preliminary Design Value
C T Tank (Post-Chlorine)		
Type	Baffled, Buried Concrete	
City of Sacramento Flow Rate	mgd	145
Number of Basins	no.	1
Volume, City of Sacramento Basin	mg	2.5
Max Water Depth	ft	16
Area Basin 1	acres	0.48
CLEARWELL		
Type	Buried Concrete, Rectangular	
Number	no.	2
Volume, City of Sacramento Clearwell	mg	16.5
Volume, PCWA, Roseville, SSWD Clearwell	mg	9
Volume, total	mg	25.5
Max Water Depth	ft	16.0
Area, City of Sacramento Clearwell	acres	3.2
Area, PCWA, Roseville, SSWD Clearwell	acres	1.7

Key:
cfs – cubic feet per second
CT – chlorine contact time
cu ft – cubic feet
ft – feet
gpm – gallons per minute
gpm/sf – gallons per minute per square feet
min – minute
mg – milligram
mgd – million gallons per day
mm -millimeter
PCWA – Placer County Water Agency
scfm/sq ft – standard cubic feet per minute per square foot
sec-1 – per second
SSWD – Sacramento Suburban Water District

5.4.2. Description of Conventional Filtration Facilities

All of the conventional filtration facilities would be constructed of concrete and painted. The grit basin, flocculation and sedimentation basins, and filters would be open-water areas. Clearwells would be covered and buried. The operations and maintenance building, chemical building, electrical building, and treated water pump station all would be enclosed buildings. These may be constructed of concrete masonry units (CMU) or steel. CMU buildings may be faced with materials such as stucco or split-face block. Steel structures would be painted.

Results of the geotechnical characterization, presented in **Chapter 2**, found that the area where the North Natomas WTP may be constructed likely has low-density granular soils and a high groundwater table. Geotechnical conditions will require more detailed evaluation in the preliminary design phase of the project to determine if the major structures would require pile foundations to prevent settling, loss of foundation support, buoyancy, or lateral spreading of soils. Also, it is expected that large amounts of dewatering would be required during construction, especially related to the buried clearwells.

5.4.2.1. Grit Basin

The purpose of a grit basin is to remove grit, such as silt and sand, to protect mechanical equipment, and prevent the accumulation of grit in the flow split, flash mix, and pretreatment processes. The grit basin is a simple sedimentation tank that removes solids via gravity settling. The grit basin would be located at the influent to the water treatment facility. Multiple basins would allow for draining, cleaning, or repair while maintaining plant operations. The basins would be rectangular, with similar configuration to the horizontal-flow sedimentation basins for improved flow characteristics. The basins would have a length-to-width ratio of nearly 5:1, with a length-to-depth ratio of greater than 15:1 to ensure good settling characteristics. At maximum plant flow, detention time would be approximately 13 minutes and the surface loading rate would be less than 7 gallons per minute per square foot (gpm/sf). Grit would be collected using chain and flight. A preliminary plan and section is shown in **Figure 5-8** (end of chapter)**.**

5.4.2.2. Flash Mix

The purpose of flash mixing is to introduce and disperse the primary coagulant chemical in raw water quickly and evenly. Complete and instantaneous dispersion of chemical coagulants is necessary to achieve optimum coagulation and flocculation, and to maximize the use of the coagulant. Aluminum sulfate (alum) requires a mixing time of less than 1 second. The amount of energy required to achieve mixing is described by the velocity gradient (G). A G value of 750 to 1,000 sec^{-1} is typically required to achieve proper initial mixing. Energy can be input to the water either mechanically or hydraulically.

A pumped diffusion injection mixing system is a hydraulic method for flash mixing that is recommended for the North Natomas WTP. This mixing system achieves dispersion of the coagulant by diverting a portion of the mainstream flow through a flash mix pump, and then injecting the chemical on the discharge side of the pump in the immediate vicinity of the counter-current injection nozzle. Velocities in the injection nozzle are designed to be in the range of 25 to 30 fps. This velocity provides nearly instantaneous dispersion of the coagulant. This system is advantageous because it requires minimal energy input, provides efficient use of coagulant, causes little headloss, and has low operation and maintenance costs. Also, it is effective over a wide range of plant flows.

5.4.2.3. Flocculation and Sedimentation

The objective of flocculation is to induce contacts between coagulated particles formed in the flash mix process by providing gentle and prolonged agitation; the particles collide, forming larger and more easily settling floc. The sedimentation process removes suspended particles heavier than water by gravity settling. Flocculation and sedimentation basins vary in configuration, mixer type, baffling design, and sludge removal equipment. The width and depth of the flocculation basins should match the sedimentation basins.

Flocculation basins are sized by the required detention time. Typical detention times range from 20 to 40 minutes depending on the source water, coagulant used, and downstream treatment provided. A detention time of at least 25 minutes is recommended for the North Natomas WTP. Flocculators can be configured either horizontally or vertically. Vertical shaft flocculators are recommended for the North Natomas WTP to minimize the impact of a failed drive unit and allow for easier inspection and preventative maintenance of motors and gearboxes. The fundamental design parameter for mechanical flocculators is the velocity gradient, G. Typical values of G range from 15 to 60 sec^{-1}. Normal practice is to taper the flocculation; that is, reduce the G value as the flow proceeds through the flocculation basin.

To transition floc particles smoothly into the sedimentation basin, there would be a diffuser wall between the flocculation and sedimentation basin. This would allow for a smooth hydraulic transition that prevents floc breakup.

Sedimentation basins have several design criteria, including detention time, surface loading rate, and effective water depth. Sedimentation basins can be configured as horizontal-flow basins, circular clarifiers, or solids contact basins. Horizontal-flow basins are recommended for the North Natomas WTP because they are the most flexible for the highly variable source water quality that could be seen at this facility. Sedimentation detention time can range from 90 to 180 minutes; a minimum of 120 minutes is recommended for the North Natomas WTP. Surface loading rates should be 1 gpm/sf or less. Water depth is typically 12 to 18 feet; 16 feet is recommended for the North Natomas WTP. Sludge collection for horizontal-flow basins can be either chain and flight with cross collection or a traveling bridge mechanism; chain and flight is recommended for the North Natomas WTP. A preliminary plan and section is shown in **Figures 5-9** through **5-11** (end of chapter).

5.4.2.4. Filtration

Filtration is a physical and chemical separation process to remove suspended and colloidal materials from water by passing the water through a porous medium. Filters have several design criteria, including filtration rate, size and number of filters, media selection, backwash system, and underdrain type. Gravity filters with constant level and constant rate are proposed for the North Natomas WTP.

It is recommended that a maximum filtration rate of 6 gpm/sf, with one filter out of service, be used for the North Natomas WTP. This would require 28,420 square feet of filter area at 235 mgd. Each filter area should be less than 1,600 square feet; therefore, 24 filters, with an area of 1,280 square feet each, are recommended. Each filter would consist of two bays, with each bay being 16 feet wide and 40 feet long.

Filters can have single-, dual-, or tri-media. Given the expected source water quality, and proposed pre-treatment processes, dual media filters are recommended for the North Natomas WTP. These filters would comprise 30 inches of anthracite coal and 12 inches of sand.

Numerous types of filter underdrains are available. A false filter bottom with nozzles is recommended for the North Natomas WTP to allow for uniform distribution of backwash flow.

The North Natomas WTP would be equipped with an auxiliary backwash system, consisting of air scour wash. The filters also would be equipped with piping to allow for filter-to-waste after backwashing. This would allow lower quality water produced during filter maturation to be sent to the equalization basins prior to recycling to the headworks. A preliminary plan and section are shown in **Figure 5-12** (end of chapter).

5.4.2.5. Future Ultraviolet Light

To comply with potential future requirements of the LT2ESWTR, water treatment design for the North Natomas WTP includes adequate footprint and hydraulic head for future UV light installation. USEPA recommends that chlorination occur upstream from UV treatment, but it can occur downstream, as proposed for the North Natomas WTP. The footprint for the 235 mgd facility is approximately 11,000 square feet plus access area, assuming the use of seven low pressure reactors with one standby unit. Maximum overall head loss for a UV system is estimated at 8 feet. However, most literature cites 3 feet as a required hydraulic standard. Initial design layout would include a head loss of 6 feet for future UV installation.

Several design parameters must be considered during the preliminary design process for UV installation. Water quality, UV lamp fouling/aging, chemical considerations and application points, flow rate, and power quality are the primary constituents that drive validation of the UV system. These parameters would be investigated during the preliminary design phase of the project to ensure that the final design was compatible with all UV operational requirements.

5.4.2.6. Chlorine Contact Tank and Clearwell

The North Natomas WTP would have two treated water clearwells, one to service Sacramento and the other to service the remaining SRWRS partners. Two clearwells were chosen because treated water pumping is expected to vary between Sacramento and its partners. The volume of the clearwells has been set at approximately 10 percent of the North Natomas WTP capacity (25.5 million gallons (MG)), with 16.5 MG dedicated to Sacramento and 9 MG dedicated to the remaining SRWRS partners. This volume is intended to provide operational flexibility at the North Natomas WTP, but would not accommodate peaking flows to the SRWRS partners' distribution systems. An intertie would be located between the clearwells that would remain closed except during emergencies and maintenance.

Since conventional filtration would be implemented for the North Natomas WTP, disinfection would likely be required to achieve 0.5-log inactivation of *Giardia*, and 2-log inactivation of viruses to meet the 3/4-log reduction for *Giardia*/viruses. Inactivation requirements for *Giardia* are significantly higher than for viruses, so meeting the 0.5-log inactivation for *Giardia* would govern.

Inactivation must be completed prior to distribution to the first customer. Sacramento could have future customers located near the North Natomas WTP; therefore, all disinfection requirements must be met prior to distribution. For this reason, an additional CT tank, with a volume of 2.5 MG, was added to the treated water train for Sacramento. PCWA could have future customers located near the Placer County line; therefore, all disinfection must be met at that point in the pipeline. SSWD plans to serve its first customer near the intersection of Walerga and Antelope roads, and Roseville would take delivery at a potable water tank site adjacent to the Pleasant Grove Waste Water Treatment Plant.

The North Natomas WTP would use free chlorine as the primary and secondary disinfectant. Chlorine contact time (CT) would be calculated to determine the required inactivation. The amount of CT required can be estimated using the following equation:

$$CT_{req'd} = 0.2828 \text{ x } pH^{2.69} \text{ x Residual Chlorine Concentration (mg/L)}^{0.15} \text{ x (log reduction required) x } 0.933^{(\text{Temperature [in C]}-5)}$$

From this equation, it can be seen that increases in pH and residual chlorine concentration cause an increase in the CT required, while an increase in temperature causes a reduction in required CT. **Table 5-12** summarizes a range of potential CT requirements for the North Natomas WTP.

Table 5-12 Potential CT Requirements for the North Natomas WTP

Temperature (°C)	Chlorine Residual (mg/L)	Log Inactivation Required	CT Required (mg/L-min) at pH = 6	CT Required (mg/L-min) at pH = 6.5	CT Required (mg/L-min) at pH = 7
7	0.5	0.5	14	17	21
7	0.5	1.0	27	34	42
7	1.0	0.5	15	19	23
7	1.0	1.0	31	38	46
7	1.5	0.5	16	20	25
7	1.5	1.0	32	40	49
22	0.5	0.5	5	6	7
22	0.5	1.0	10	12	15
22	1.0	0.5	5	7	9
22	1.0	1.0	10	14	16
22	1.5	0.5	6	8	9
22	1.5	1.0	12	14	17

Key:
°C – degrees Celsius
CT – chlorine contact time
mg/L-m – milligrams per liter per minute
mg/L – milligrams per liter
WTP – water treatment plant

Since predisinfection would occur at the headworks, some attributable CT would be achieved in the grit basins, flocculation/sedimentation basins, filters, and miscellaneous piping. The majority of CT would be achieved in the CT tank (Sacramento only) and the clearwells, with additional credit available in the treated water piping (PCWA, SSWD, and Roseville only). The amount of CT achieved is calculated using the following equation:

$$CT_{ach} = \text{Residual Chlorine Concentration, mg/L x}$$
$$(T_{10}/T \text{ x [Basin Volume, gallons / Plant Flow Rate, gpm])}$$

The T_{10}/T ratio compares the disinfection contact time to the theoretical detention time in a basin. This ratio can be determined by the baffling classification in a basin. The CT tank and clearwells would be designed to provide T_{10}/T ratios of 0.7. The design would include perforated inlet baffles, serpentine or perforated intra-basin baffles, and either an outlet weir or perforated launders. Pipelines are assigned T_{10}/T ratios of 1.0 for perfect plug-flow conditions. The delivery turnouts will be designed to include the equipment necessary to calculate the CT_{ach} in the pipelines. **Table 5-13** summarizes the CT that would be achieved at the North Natomas WTP under a difficult case scenario, low temperature (7 °C), high pH (7 units) water with 1-log inactivation required.

Table 5-13 CT Achievements for 1-log Inactivation Requirements

Residual Chlorine (mg/L)	T_{10}/T	$CT_{req'd}$ (mg/L-min)	Sacramento Flow (mgd)	Sacramento CT tank/ Clearwell Volume (MG)	Sacramento CT_{ach} (mg/L-min)	Minimum Sacramento Clearwell Volume to Achieve CT On-site (%)	Partners Flow (mgd)	Partners Clearwell Volume (MG)	Partners CT_{ach}, (mg/L-min)
0.5	0.7	42	145	19	66	83	90	9	64
1.0	0.7	46	145	19	132	46	90	9	35
1.5	0.7	49	145	19	198	32	90	9	25

Key:
CT – chlorine contact time
CTach – chlorine contact time achieved
MG – million gallons
mg/L - min – milligrams per liter per minute
mg/L – milligrams per liter
mgd – million gallons per day

This table shows that at maximum plant flow, CT requirements can be met for a difficult case scenario such as low temperature and high pH, and at a variety of residual chlorine concentrations. The size of the clearwells also provides some flexibility for operating levels, allowing the clearwells to be at varying levels while still meeting CT on site. Worst case conditions occur under low chlorine residual concentrations, less than 0.5 mg/L, when CT achieved upstream of the clearwells and in the treated water piping could have a significant impact on the overall CT achieved. Preliminary plan drawings are shown in **Figures 5-13** and **5-14**.

5.4.2.7. Operations and Administration Building

Efficient management of any WTP is highly dependent on the design and layout of the operations and administration building. This building serves as the major human interface between the WTP and its operators. To design and, ultimately, build an effective building, the overall building design must address the functionality and architectural and structural integrity of the structure.

Functional design components can be divided into four spatial categories: (1) administrative zone, (2) operational management zone, (3) product quality control zone (laboratory), and (4) the mechanical/workshop zone. The administrative section is critical for day-to-day operations of the plant and may consist of a reception area, storage room for records and office supplies, toilet facilities, a conference room, and offices for plant managers. The operational management zone serves as an interface between staff and the process operations of the treatment plant. A control room, laboratory, and a lunchroom are essential components of the management area. If sampling for water quality control is done on-site, the laboratory should consist of four discrete areas: general chemistry lab, instrumentation lab, bacteriology lab, and a management office. Each laboratory area would have specific design requirements that would help achieve successful water quality management. The fourth spatial area is the mechanical/workshop zone. This area should house the building's mechanical equipment and provide adequate working space for computer or electronic repair.

The proposed operations and administration building for the North Natomas WTP is a two-story structure, with a footprint of 10,000 square feet. A preliminary plan and elevation are shown in **Figures 5-15** and **5-16**. Pursuant to client request and efficient plant management, the proposed building parking lot is located directly ahead of the entrance to the plant. In addition, plant access driveways direct all public traffic to the building. The entrance design and building locale discourage unauthorized vehicles from entering the site and allow the plant staff to control visitor traffic.

The architectural and structural aspects of the building are highly dependent on owner preference and Federal, State, and local laws. Coordination between the owner and engineering design team would prove to be invaluable for the successful architectural and structural design of the building. It is anticipated that the building would be constructed of CMU with a stucco facing. Colors would be selected to blend with the expected urban development in the North Natomas area.

5.4.2.8. Treated Water Pump Station

Two treated water pump stations would be built at the North Natomas WTP. A pump station would be located on each clearwell, with one servicing Sacramento and the other servicing PCWA, SSWD, and Roseville.

The Sacramento pump station total design flow would be 145 mgd with a design total dynamic head of 161 feet. The pump station would likely consist of six pumps, two at approximately 18 mgd and four at approximately 36 mgd capacities at the design head with some equipped with variable frequency drives. One 36-mgd pump would be a standby pump. The total connected horsepower, including the backup pump, would be approximately 6,400 hp.

The PCWA, SSWD, and Roseville pump station design flow would be 90 mgd with a design total dynamic head of 419 feet. The pump station would likely consist of six pumps, two at 11 mgd and four at 22.5 mgd at the design head. Two or three of these pumps would be equipped with variable frequency drives to accommodate turndowns. One 22.5-mgd pump would be a standby pump. The total connected horsepower, including the backup pump, would be approximately 9,950 hp.

One switchgear and control building would be built for the pump stations. The building would be constructed on grade and adjacent to the clearwells and would measure at least 75 feet by 25 feet.

5.4.2.9. Plant Hydraulics

Raw water would be delivered to the grit basin by the raw water intake pump station. Water would then flow by gravity through the flocculation/sedimentation basins and filters to the treated water clearwells.

The facilities within the site would be positioned to maximize process flow efficiency and eliminate the need for booster pumps between the treatment processes. Although the topographic layout of the site is generally level, material excavated for the stormwater detention basins, equalization basins, and sludge setting basins may be used as fill to raise other facilities. This would increase the water surface elevation and improve plant hydraulics, and reduce or eliminate the need to haul excavated soil off site. **Tables 5-14** through **5-16** summarize the estimated hydraulics through the plant and **Figures 5-17** and **5-18** show hydraulic profiles of the facilities at three potential WTP sites. **Table 5-14** and **Figure 5-17** represent conditions at a site located at the western end of potential WTP sites and is assumed to have an area of approximately 90 acres. **Table 5-16** and **Figure 5-18** represent conditions at a site located at the eastern end of potential WTP sites and is assumed to have an area of approximately 100 acres. **Table 5-15** represents conditions at a site located near the middle of potential WTP sites and is assumed to have an area of approximately 100 acres. The sites located at the western end and near the middle of potential sites will have the same finished grade and water surfaces; therefore, **Figure 5-17** represents the hydraulic profile for both sites. It should be noted that the existing grade at the western and middle sites is not located within the 100-year floodplain. However, the site at the eastern end of potential sites would be located within the 100-year floodplain, as per Flood Insurance Rate Maps (FIRM) dated July 1998. Raising the grade may minimize or eliminate the flood concern at this site. This should be considered during the engineering analysis as part of the WTP site selection.

Table 5-14 North Natomas WTP Site at Western End of Potential Sites – Hydraulic Summary

Facility	Existing Grade (ft)	Finished Grade (ft)	Water Surface Elevation (ft)	Max. Depth (ft)	Head Loss (ft)
Grit Basin	22.5	33.0	45.0	12	2.0
Flash Mix	23.2	29.5	-	-	1.7
Flocculation/ Sedimentation	23.7	29.5	41.3	16	1.5
Filters	23.6	28.5	39.8	14	9.2
UV Disinfection (future)	24.2	24.2	-	-	8.0
CT/Clearwell	23.5	23.5	22.6	16	2.6
Equalization Basins	23.3	23.3	20.0	15	-
Sludge Settling Basins	22.5	22.5	20.0	4	-

Key:
CT – chlorine contact time ft – feet UV – ultra violet WTP – water treatment plant

Table 5-15 North Natomas WTP Site Near Middle of Potential Sites – Hydraulic Summary

Facility	Existing Grade (ft)	Finished Grade (ft)	Water Surface Elevation (ft)	Max. Depth (ft)	Head Loss (ft)
Grit Basin	20.2	33.0	45.0	12	2.0
Flash Mix	20.0	29.5	-	-	1.7
Flocculation/ Sedimentation	20.0	29.5	41.3	16	1.5
Filters	20.0	28.5	39.8	14	9.2
UV Disinfection (future)	19.9	24.0	-	-	8.0
CT/Clearwell	19.8	23.5	22.6	16	2.6
Equalization Basins	19.9	23.0	20.0	15	-
Sludge Settling Basins	20.0	22.5	20.0	4	-

Key:
CT – chlorine contact time ft – feet UV – ultra violet WTP – water treatment plant

Table 5-16 North Natomas WTP Site at Eastern End of Potential Sites – Hydraulic Summary

Facility	Existing Grade (ft)	Finished Grade (ft)	Water Surface Elevation (ft)	Max. Depth (ft)	Head Loss (ft)
Grit Basin	15.0	28.0	40.0	12	2.0
Flash Mix	15.0	24.5	-	-	1.7
Flocculation/ Sedimentation	15.0	24.5	36.3	16	1.5
Filters	15.0	23.5	34.8	14	9.2
UV Disinfection (future)	15.0	19.0	-	-	8.0
CT/Clearwell	15.0	18.0	17.6	16	2.6
Equalization Basins	15.0	15.0	15.0	15	-
Sludge Settling Basins	15.0	15.0	15.0	4	-

Key:
CT – chlorine contact time ft – feet UV – ultra violet WTP – water treatment plant

5.4.3. Description of Solids Handling Facilities

Waste streams generated at the North Natomas WTP include grit from the grit basin, sludge removed from the sedimentation basins, filter backwash water, filter-to-waste water, and sampling water.

Filter backwash water, filter-to-waste water, sampling water, and sludge settling basin decant water would be treated with a polymer and then stored in an equalization basin. The basin would be sized to accommodate four filter backwashes and filter-to-waste cycles plus 10 percent. Two basins would be provided to allow for cycling. Decant would be recycled to the headworks while the solids would be sent to the sludge settling basins. **Table 5-17** summarizes the design of the equalization basins.

Table 5-17 Design Criteria for the Equalization Basins

Description	Units	Preliminary Design Value
Backwash Volume Required	cu ft	144,500
Number of Basins	no.	2
Water Depth	ft	15
Basin Width	ft	40
Basin Length	ft	120
Volume, each	cu ft	72,000
Volume, total	cu ft	144,000

Key:
cu ft – cubic feet ft – feet no. – number

Solids from the grit basin and the equalization basin and sludge from the sedimentation basin would be sent to sludge settling basins for drying. The facilities would be located to allow for cycling of drying beds on 4-month cycles. Solids generation was estimated for the 4-month winter period from December through April and the remaining 8 months of the year, hereafter called summer. It has been assumed that 10 pounds of solids can be applied per square foot for evaporative drying during the winter months and 15 pounds per square foot during the summer. Six settling basins would be provided to allow for cycling and settling periods. Three of these settling basins would be dedicated as winter settling basins and three would be used during the remainder of the year. The quantity of solids generated by these waste streams was estimated using the equation below based on the coagulant dose, polymer dose, treatment flow, and total solids in the raw water. **Table 5-18** summarizes the estimated solids generation.

Solids Production Rate (lbs/day) =

[(Alum Dose in mg/L x 0.26) + (Turbidity in NTU x 1.2) + Polymer Dose in mg/L](8.34)(Flow in mgd)

Table 5-18 Estimated Solids Generation for the North Natomas WTP

Average Parameter	Winter Conditions (December – March)	Summer Conditions (April – November)
Average Flow (mgd)	120	185
Raw Water Turbidity (NTU)	47	25
Alum Dose (mg/L)	40	20
Polymer Dose (mg/L)	0.7	0.7
Average Solids Production Rate (lbs/day)	**67,500**	**55,400**

Key:
lbs/day – pounds per day
mgd – million gallons per day
mg/L – milligrams per liter
NTU – nephelometric turbidity units
WTP – Water Treatment Plant

To size the sludge settling basins, several criteria were evaluated. First, settling basin loading rates described above were used to calculate the area required for each bed. Then, a depth of sludge was determined. Typically, settling basins should be less than 6 feet in depth. The total volume of solids produced per period was calculated assuming 4 percent solids sludge when drying, which is expected to have a density of approximately 64 pounds per cubic foot. The total area of the settling basins was then used to identify the required sludge depth. **Table 5-19** lists the design criteria for the sludge settling basins.

Table 5-19 Design Criteria for Sludge Settling Basins

Description	Units	Preliminary Design Value
Target Sludge Settling Basin Loading	lb DS/ft^2	10/15
Number of Settling Basins (winter/summer)	no.	3/3
Settling Basin Width	ft	275
Settling Basin Length	ft	1100
Settling Basin Depth (winter/summer)	ft	4/5.5
Total Settling Basin Volume	ft^3	9,075,000

Key:
ft3 – cubic feet
ft – feet
lb DS/ft2 – pounds dry solids per square foot
no. – number

The settling basins would be designed with a downward slope of 0.5 to 1.0 percent toward the outlet, with a vehicle access ramp. A multilevel decant facility would be built that would operate continuously during the drying cycle. The decant water would be continuously returned to the equalization basins. Consideration could be given to the use of polymers or aeration in the sludge settling basins during preliminary design phase of the project.

It should be noted that the assumed 90-acre WTP site at the western end of potential sites has inadequate space for all of the required settling basins; only five settling basins have been shown on the layout. It may be possible to increase the depth of the settling basins to enhance the overall capacity, but this may be at the detriment of settling basin performance. The assumed 100-acre WTP sites at the middle and eastern end of the potential sites would be able to provide better configuration and access to the settling basin.

Dried sludge would be transported to a landfill for ultimate disposal. It is expected that settling basins would be routinely cleaned, and dried sludge removed approximately three times per year.

5.4.4. Description of Chemical Feed and Supply Systems

The North Natomas WTP would include chemical feed and storage systems for the chemicals shown in **Table 5-20**. Chemical application points are shown in **Figure 5-19**.

Table 5-20 Summary of Chemicals Selected and Purpose

Chemical	Purpose	Injection Point
Chlorine (Cl_2)	CT Disinfection Credit	Raw Water Line, Downstream from Recycle Line
Potassium Permanganate (KMnO4)	Taste and Odor – Rice Herbicides	Raw Water Line, Downstream from Recycle Line
Aluminum Sulfate (Alum)	Coagulation	Flash Mix Pump Discharge
Polyaluminum Chloride (PACl)	Coagulation	Flash Mix Pump Discharge
Cationic Polymer	Coagulation Aid	Flash Mix Pump Discharge
Sodium Hydroxide (NaOH – Caustic Soda)	pH Adjustment	Flash Mix Pump Discharge
Anionic Polymer	Flocculation Aid	Flocculation Basin Influent Channel
Chlorine (Cl_2)	CT Disinfection Credit	Filter Influent Channel
Non-Ionic Polymer	Filter Aid	Filter Influent Channel
Chlorine (Cl_2)	CT Disinfection Credit	CT Tank Influent Box (Sacramento) Clearwell Influent Channel (PCWA, Roseville, SSWD)
Hydrofluosilicic Acid (H_2SiF)	Fluoridation	CT Tank Influent Box (Sacramento only)
Quicklime (CaO)	pH Adjustment	Treated Water Pump Intake Channel

Key:
CT – chlorine contact time PCWA – Placer County Water Agency SSWD – Sacramento Suburban water District

5.4.4.1. Chlorine Gas

Chlorine is obtained as a pressurized gas in 1-ton cylinders. The chemical would be fed into the raw water line, filter influent channel, and filter effluent weir. Six chlorinators would be provided, one for each feed location and one as a spare. The chlorine system would include cylinder-mounted vacuum regulators, scales, automatic switchover system, chlorinators, injectors, leak detectors, and associated piping, valves, and controls. Feed and storage equipment would be located in a chemical building. Adequate space would be provided for moving the cylinders with an overhead hoist and trolley system. Ventilation would be provided in both the storage and feed areas. Storage would be provided for 30 days.

5.4.4.2. Potassium Permanganate

Potassium permanganate is obtained in dry, granular form in pails or drums. The chemical would be fed into the raw water line. The feed system would include a volumetric feeder with hopper for loading chemicals. The permanganate would be fed into solution tanks that would use raw water. The solution would then be injected into the raw water line. It is recommended that feed equipment be located in an adjacent building adjacent, with a ventilation system, since the chemicals are very heavy and difficult to handle. Since this system would not be used often, only 7 days of storage would be provided.

5.4.4.3. Aluminum Sulfate

Aluminum sulfate (alum) is obtained as a liquid (49 percent solution) in bulk delivery. This chemical would be fed into the flash mix. Diaphragm metering pumps would deliver the coagulant to the flash mix area. Three metering pumps would be provided for each of the three process trains, two for feed and one for standby. A magnetic flow meter would be used on the discharge delivery piping as near to the point of application as possible for feedback control of the metering pumps. The storage tanks would be located on site and the feed equipment would be located in a chemical building, in a sealed room with a ventilation system. Thirty days of storage would be provided.

5.4.4.4. Polyaluminum Chloride

Polyaluminum chloride (PACl) is an alternate primary coagulant that would likely be used during the winter months when the raw water turbidity is higher and pH range can vary more widely. Liquid can be obtained as a 50 percent solution. Two metering pumps would be provided for each of the three process trains: one duty pump and one standby pump. A magnetic flow meter would be used on the discharge delivery piping as near to the point of application as possible for feedback control of the metering pumps. The storage tanks would be located on site and the feed equipment would be located in a chemical building in a sealed room with a ventilation system. Seven days of storage would be provided.

5.4.4.5. Cationic Polymer

Since the dosing for cationic polymer can be variable, it is not desirable to store large volumes. Cationic polymer is obtained in liquid form (100 percent active) in 300-gallon bins. Two metering pumps would be provided for each of the three process trains. The feed equipment would be located in a chemical building, in a sealed room with ventilation system. Fourteen days of storage would be provided.

5.4.4.6. Caustic Soda

Caustic soda is obtained as a liquid (25 percent solution) in bulk delivery. The chemical would be fed into the flocculation influent channel as needed for pH adjustment. Diaphragm metering pumps would deliver the caustic soda to the flocculation influent channel. The feed equipment would be located in a chemical building in a sealed room with a ventilation system. Since this system would not be used often; only 7 days of storage would be provided.

5.4.4.7. Anionic Polymer/Nonionic Polymer

Many types of these polymers are available, which can be provided in dry or liquid form. These polymers are usually added at very low doses, making storage and feed systems relatively small. A package polymer feed system would be planned for feeding either dry or liquid form that includes dry feeder, mixing tank, aging tank, and metering pumps. Space would be provided for 14 days of storage for either barrels or pallets.

5.4.4.8. Hydrofluosilicic Acid

Hydrofluosilicic acid is obtained as a liquid (23 percent solution) in bulk delivery. This chemical would be fed only into the water delivered to Sacramento. For this reason, it would be fed into the Sacramento CT tank influent channel. This chemical is highly corrosive, even when diluted, and therefore needs to be located near its point of application. Storage and feed equipment need to be constructed of specific materials to resist corrosion. Storage needs to be 100 percent contained for maximum acid volume. Space would be provided for 30 days of storage.

5.4.4.9. Lime

Lime is added to treated water as a corrosion control measure to elevate pH and add alkalinity. The pH adjustment alone is not sufficient for a low buffering capacity water, such as in the Sacramento River. Lime is added to obtain a positive Langelier Index (to maintain excess calcium carbonate in the treated water). Lime adds calcium to the water, unlike caustic soda or soda ash, which then can precipitate and be deposited on pipe walls to enhance corrosion control. Lime is available in two forms: quicklime and hydrated lime. Hydrated lime is more expensive than quicklime and more needs to be added to provide the same corrosion control as quicklime. Storage facilities for hydrated lime also need to be larger, but the feed equipment is easier to operate and maintain. Quicklime is recommended due to space and cost efficiency, and requires storage facilities and a slaker to create the lime slurry for feeding. Since lime slurry is difficult to pump, storage and feed facilities should be as close to the point of application as possible. Space would be provided for 30 days of storage of quicklime.

5.4.4.10. Chemical Buildings

Lime and fluoride should be housed together, located adjacent to the Sacramento CT tank and clearwells. See **Figure 5-20** for a floor plan and elevation of this building.

Chlorine gas and caustic soda should be housed together since caustic soda is used to scrub chlorine leaks. All other chemical storage and feed systems also could be stored in this building. This building should be centrally located to reach all chemical application points, but also be located near the operations building to allow for frequent visits by operation and maintenance staff. See **Figure 5-21** for a plan and elevation of this building.

All feed equipment and storage facilities should be enclosed in buildings. **Table 5-21** summarizes the chemical feed and storage requirements.

Table 5-21 Summary of Chemical Feed and Storage Requirements

Chemical	Storage Criteria	Storage Weight or Volume	Type of Container	Number of Containers
Chlorine Gas	30 days @ 2.5 mg/L and 235 mgd	147,000 pounds	2,000-pound cylinders	80 cylinders
Potassium Permanganate	7 days @ 1.5 mg/L and 195 mgd	184 cubic feet	Vertical steel hopper	1 - 200 cubic feet
Aluminum Sulfate	30 days @ 15 mg/L and 235 mgd	164,000 gallons	Vertical steel, rubber-lined	5 – 35,000 gallons
Polyaluminum Chloride	7 days @ 0.4 mg/L as Al and 145 mgd	6,240 gallons	Vertical steel, rubber-lined	1 – 7,000 gallons
Cationic Polymer	14 days @ 0.5 mg/L and 235 mgd	1,520 gallons	300-gallon bins	6 bins
Caustic Soda	7 days @ 5 mg/L and 90 mgd	9,730 gallons	Horizontal steel	2 – 5,000 gallons
Anionic/ Nonionic Polymer	14 days @ 0.2 mg/L and 235 mgd	5,500 pounds	50-pound bags or 55-gallon drums	110 bags or 11 drums
Hydrofluosilicic Acid	30 days @ 0.8 mg/L as F and 145 mgd	16,000 gallons	Fiberglass	2 – 9,000 gallons
Lime	30 days @ 5 mg/L and 235 mgd	5,770 cubic feet	Vertical steel silos	1 – 6,500 cubic feet

Key:
al – aluminum mgd – million gallons per day F – Fluoride mg/L – milligrams per liter

5.4.5. Electrical Feed and Supply Considerations

This section presents a discussion of primary power requirements, power availability, and power reliability for the North Natomas WTP. Also discussed are motor starter requirements, primary backup power and supply, and an alternative backup power supply option. A description of the electrical building is also included.

5.4.5.1. Primary Power Requirements, Availability, and Reliability

The maximum power requirement for the 235 mgd North Natomas WTP has been estimated to be 17,850 kVA. **Table 5-22** generally summarizes how the power requirements were estimated.

Table 5-22 Power Requirement Summary

Facility	Peak Flow (mgd)	Pump (hp[1])	Misc. Load (kVA)	Power (kVA)	Amps @ 4,160 Volts	½ Load (kVA)
Sacramento	145	6,400	1,500	17,850	2,480	8,925
PCWA/Roseville/ SSWD	90	9,950				

Notes:
[1] Includes spare pump.
Key:
hp – horsepower
kVA – kilovolt-ampere
mgd – million gallons per day
PCWA – Placer County Water Agency
SSWD – Sacramento Suburban Water District

SMUD is the governing power utility for the potential WTP sites. Power for this load would be available from existing SMUD lines routed along Elverta Road up to Power Line Road. Two 69 kV power lines (in parallel) are currently in place and SMUD is in the process of upgrading these lines due to increased commercial and residential development in the North Natomas area. The loads presented here can be considered in the SMUD upgrade. The various potential WTP sites would be supplied with power as follows (see **Figure 5-22**).

- WTP site located at the western end of potential sites: This site would be located approximately 1 mile east of the Garden Highway. Power feed for this site would be supplied from existing lines on Elverta Road up to Power Line Road. The power feed would continue underground to the WTP location. It is expected that this line may have to be routed underground due to safety concerns with aboveground facilities located in the approach and departure zone for Sacramento International Airport. These lines would be located within the Elverta Road right-of-way. SMUD indicated that underground 69 kV lines have a budget cost of $175.00 per foot (not including trenching). SMUD requires the owner to incur the cost of poles and trenching for underground lines.
- WTP site located in the middle of potential sites: This site would be located approximately 1 mile east of Power Line Road. This 69 kV service is available from upgraded existing overhead lines along Elverta Road. SMUD indicated that overhead 69 kV feed from its line to a transformer has a budget cost of $30.00 per foot.
- WPT site located at the eastern end of potential sites: This site would be located just east of Highway 99. This 69 kV service is available from upgraded existing overhead lines along Elverta Road. SMUD indicated that overhead 69 kV feed from its line to a transformer has a budget cost of $30.00 per foot.

SMUD can provide a design that would incorporate the level of redundancy the owner would require. SMUD can design its connection points and multiple switching configurations for the redundancy that would meet the needs and satisfaction of the owner.

5.4.5.2. Motor Starter Requirements

SMUD requires all large medium-voltage, and all large low-voltage motors to have reduced-voltage solid state starters.

5.4.5.3. Primary Backup Power Supply

The proposed primary means for backup power supply is installing two primary feeds at the North Natomas WTP site. The reliability of power supply at the North Natomas WTP site would increase greatly with installation of these two separate primary feeds into the two transformers at the North Natomas WTP to feed 4.16 kV into the power distribution substation. The proposed plan for the power feeds at the North Natomas WTP is to receive one feed from each of the two existing upgraded parallel 69 kV lines into the site.

Each secondary transformer would be connected to a main circuit breaker. The two mains would be connected by a tiebreaker. Upon loss of power detected in one of the two main breakers, that main would open and after a specified time delay (selected by the owner), the tiebreaker would close, resuming power to the side of the bus that lost power.

5.4.5.4. Alternative Backup Power Supply Option

SMUD does not allow another utility to serve within the SMUD service area. An alternative backup power supply option would be use of diesel generators at the North Natomas WTP site. The SRWRS partners selected a 50 percent backup generation capacity for evaluation. The required 50 percent backup generation for the 235 mgd North Natomas WTP site would require 8,925 kVA of paralleled generators. The parallel generators would require a switchgear for control, a day tank (300 gallons) for each generator, and a fuel storage tank. The generators use 150 gallons of fuel per hour at full load and would require a total of 6,000 gallons of fuel for an 8-hour time period (full load). The paralleling switchgear would control the output power for each generator; therefore, if the load was less than 8,925 kVA, fuel consumption would be less.

The space required for the paralleling switchgear, low voltage controls, fuel tank, and generators would be approximately 10,000 square feet in a building with integral automatic air flow louvers and fire alarm system design.

A more detailed evaluation of backup power requirements and specific loads that would be deemed critical if both main breakers into the plant were lost is strongly recommended during the preliminary design phase of the project to optimize sizing of these generators and associated facilities.

5.4.5.5. Electrical Building

Power would enter the site and go directly to transformers to reduce voltage from 69 kV to 4.16 kV. The secondary of the transformers would then go to two main breakers at the North Natomas WTP power distribution substation. The transformer area is expected to be 130 feet by 130 feet, per SMUD requirements. The plant substation, medium-voltage switchgear building is expected to be 50 feet by 50 feet. The building would house the two mains, tie breaker, Potential Transformers (PTs) and CPTs and each of the two buses would have a capacity of eight 4.16 kV 1,200-amp breakers. The distribution substation would have breakers to feed all of the 4.16 kV loads on the plant. Minimum building requirements would include exhaust fans and heaters.

It is anticipated that the 480-volt loads would be distributed from one or two large power centers centrally located to serve the plant's 480-volt motor control center loads.

5.4.6. Sewer and Stormwater Management

Sewer from the operations and administration building would be conveyed to the County Sanitation District-1 (CSD-1) collection system. CSD-1 plans to extend its system into the Sacramento Metropolitan Airpark. A 12-inch trunk, the NN Metro Air Trunk, is planned to be installed up to Elverta Boulevard, approximately 3,600 feet east of Power Line Road. CSD-1 was contacted and confirmed that the NN Metro Air Trunk would be able to accept the discharge of wastewater from the North Natomas WTP. It is expected that a sewer line, not to exceed 6 inches, would be installed from the WTP site to the connection with the NN Metro Air Trunk.

Currently, no storm drainage services are located in the northwest corner of Sacramento County near the project area. It has been assumed that all stormwater would need to be captured and managed on site. The site would be constructed and graded to collect stormwater runoff and channel it to on-site detention basins. These basins would be sized to meet the capacity of a 10-year storm over 5 days. The Sacramento City/County Drainage Manual indicates that the water depth of such a storm would be 5.76 inches. Overall site areas are approximately 100 acres, but open-water facilities, such as sedimentation basins and sludge settling basins, would not contribute to stormwater runoff. These areas account for approximately 30 acres. The remaining 70 acres could contribute to stormwater run off at varying rates. It was assumed that overall, 70 percent of rainfall would run off the site. It has been estimated that just over 1 million cubic feet of water would need to be planned for in the detention basin design. It was assumed that the detention basins would be 3 feet deep to allow for evaporative drying. Therefore, approximately 7 acres of detention basins would be required on site.

5.4.7. Site Configuration and Layout

Preliminary site configurations and layouts have been prepared for three representative sites in the potential WTP area (refer to **Figure 5-1**). Each of the sites includes full 8-foot fencing with victory arms. A gate would be placed at the main entry, which would be set back from Elverta Boulevard to allow trucks to exit the roadway while waiting for entry to the site. Landscaping at all three sites would include native or Xeriscape™ type plants to the extent possible. Landscaping would be laid out to improve the view from neighboring facilities.

5.4.7.1. Representative Sites Located at Western End of Potential Sites

The site area is irregularly shaped with a total land area of approximately 90 acres, including roadways, as shown in **Figure 5-23**. The site's facilities are arranged in the design to maximize process flow efficiency and to address operational, security, and FAA safety concerns.

The grit basin would be located at the southwest corner of the site since this location would provide the closest connection to the incoming raw water pipeline. The flow split and flash mix area and flocculation/sedimentation basins would be located northeast of the grit basin. Raw water would travel north through the flocculation/sedimentation basins to the northern half of the facility. The water would turn east to the filter building to provide shortened piping connections between the two facilities and keep facilities centrally located near the operations and administration building. Just southeast of the filter building space has been allocated for a future UV facility. The water would then continue south to the clearwells located along the front side of the site with the treated water pump stations. The pump stations would be located in north corners of the clearwells to be near the electrical power source. The treated water piping would run south between the clearwells and continue east into Elverta Boulevard for distribution.

The operations and administration building would be located near the middle of the plant, on a direct path from the main entrance road for improved security. Vehicles entering the property would be directed to the operations and administration building, thereby decreasing the potential for unauthorized entry. By being able to view the front entrance, plant staff would also be able to monitor incoming and outgoing traffic to and from the plant, respectively.

The main chemical building would be located to the west of the main entrance road, increasing the safety of chemical deliveries by confining the chance of a spill from chlorine or polymer delivery trucks to this area. The chemical building would also be just south of the flocculation/sedimentation basins to reduce polymer piping length, and to be in close proximity to the clearwells to reduce chlorine piping length. The lime/fluoride building would be located east of the filter building to be in close proximity to the points of application to the treated water.

The SMUD substation and electrical building would be located to the east of the lime/fluoride building, north of the clearwells. This arrangement keeps the electrical building close to the highest power requirement, the treated water pump stations at the clearwells.

The equalization basins would be located northeast of the filter building to receive the backwash water. The remaining land at the site would be used for drying lagoons and stormwater detention basins.

5.4.7.2. Representative Sites Located at Middle and Eastern End of Potential Sites

These site areas are rectangular-shaped with a total land area of approximately 100 acres, including roadways, as shown in **Figures 5-24** and **5-25**. Since these sites are of the same configuration, the layouts are very similar. The sites' facilities are arranged to maximize process flow efficiency and to address operational, security, and FAA safety concerns.

The grit basin would be located at the southwest corner of the sites since this location would provide the closest connection to the incoming raw water pipeline. The flow split and flash mix area and flocculation/sedimentation basins would be located northeast of the grit basin. Raw water would travel north through the flocculation/sedimentation basins to the northern half of the facility. The water would turn east to the filter building to provide shortened piping connections between the two facilities and keep facilities centrally located near the operations and administration building. Just southeast of the filter building, space has been allocated for a future UV facility. Water would then continue south to the clearwells located along the front side of the site with the treated water pump stations. The pump stations would be located in north corners of the clearwells to be adjacent to the electrical power source. The treated water piping would run south between the clearwells and continue east into Elverta Boulevard for distribution.

The operations and administration building would be located near the middle of the plant, on a direct path from the main entrance road for improved security. Vehicles entering the property would be directed to the operations and administration building, thereby decreasing the potential for unauthorized entry. By being able to view the front entrance, plant staff would also be able to monitor incoming and outgoing traffic to and from the plant, respectively.

The main chemical building would be located to the west of the main entrance road, increasing the safety of chemical deliveries by confining the chance of a spill from chlorine or polymer delivery trucks to this area. The chemical building would also be just south of the flocculation/sedimentation basins to reduce polymer piping length, and to be in close proximity to the clearwell to reduce chlorine piping length as well. The lime/fluoride building would be located east of the filter building to be in close proximity to the points of application to the treated water.

The SMUD substation and electrical building would be located to the east of the lime/fluoride building, northeast of the clearwells. This arrangement keeps the electrical building close to the highest power requirement, the treated water pump stations at the clearwells.

The equalization basins would be located north of the filter building to receive the backwash water. The remaining land on the sites would be used for solids drying basins and stormwater detention basins.

5.4.8. Special Considerations

The representative sites located at the western end and at the middle of potential sites are located within the overflight zone of Sacramento International Airport. For this reason, the design of these facilities must be developed to account for safety issues identified by the Sacramento County Airport Service and the FAA. Preliminary discussions with these agencies indicate that the primary area of concern is the potential for open-water areas to serve as a bird attractant, which would be undesirable for the airport.

Currently, most of the land use near the three potential sites is agricultural or rural with its primary use as rice cropping. This land use leads to an increased presence of birds in the vicinity of potential WTP sites.

For the two reasons above, it has been recognized that the water treatment detailed design would need to incorporate methods for bird detraction. Preliminary information indicates that numerous options for bird detraction exist, including the following:

- Selected design details in the buildings and facilities
- Installation of proprietary detraction devices
- Installation of false predatory birds
- Installation of a predatory bird call sound system
- Covering of open water basins

More recent discussions with the Sacramento County Airport Service and the FAA have indicated greater reluctance to accommodate facilities that may be perceived as bird attractants. Close coordination with these two agencies will be required in the time leading up to the preliminary design phase of the project.

5.4.9. Operating Characteristics

The North Natomas WTP would operate continuously, 24 hours per day, 7 days per week, at various flow rates throughout the year. At a water treatment facility of this size, operations and maintenance would be ongoing. Several types of staff would be expected on site at varying levels throughout the day, including WTP operators (16), laboratory technicians (6), electrician (1), mechanic (1), machinist (1), instrument technician (1), administrator (1), and other miscellaneous support staff (3). Most staff would be on-site during the daytime hours, from approximately 7:00 a.m. to 5:00 p.m. It is expected that WTP operators (approximately four per shift) would be on site during all hours of the day. DHS would require the North Natomas WTP to have a Treatment Grade 5 operator to supervise the operation and maintenance and Treatment Grades 2, 3, and 4 for various plant operation shifts.

Daily traffic would comprise mainly operations and maintenance staff. Specialty requirements for scheduled and emergency maintenance also would occur that may include heavier load trucks and chemical deliveries.

Numerous chemicals, as discussed previously, would be stored and used on site for water treatment operations. Primary chemicals used, including chlorine gas, aluminum sulfate, hydrofluosilicic acid, and lime, would have 30 days of storage at maximum plant daily flow. These chemicals would be delivered by large bulk transport trucks a maximum of once per month. Polymers used daily for treatment would have 14 days of storage at maximum plant daily flow. Because use of this chemical is significantly lower than the other major chemicals, delivery of polymers would occur by a smaller transport truck, a maximum of twice monthly. Other chemicals are only used seasonally, including potassium permanganate, caustic soda, and polyaluminum chloride. Therefore, delivery of these chemicals would occur only during their specific period of use, which is expected to be short.

In addition to water treatment chemicals, minor amounts of other chemicals would be used for equipment operation and operation of facilities (i.e., lubricants, oils, cleaning solvents, laboratory solutions). These would likely be stored in the operations and administration building. Diesel storage for the backup generators, if used, also would be located at the site. It has been estimated that storage would be 6,000 gallons. All chemical and fuel storage would be contained and safety procedures and best management practices would be implemented at this facility similar to other water treatment facilities of the SRWRS partners.

The treated water pump station and backup generators are proposed to be constructed inside buildings, which would reduce their noise emissions. Minor noises would occur associated with low power equipment such as sludge collectors, flocculators, and pumps, in addition to water flow noises.

5.4.10. Construction Characteristics

Construction activities would involve grading the site and erecting the new facilities described in this chapter. Because of the flat topography of the site, grading would likely occur over a large portion of the project site. This would include excavation for the clearwells and chlorine contact tank, equalization basins, sludge lagoons, and stormwater detention basins. The grit basin, flocculation/sedimentation basin, and filters would need to be raised to allow for gravity flow through the facility. It is intended that the excavated materials would be used, if acceptable from an engineering perspective, as fill on site.

Standard construction methods are proposed, but pile drivers may be used to construct footings for new water-holding structures (i.e., grit basin, flocculation/sedimentation basin, filters, etc.) if the geotechnical investigation determines they would be required. Groundwater levels are expected to be high in this area and therefore large amounts of dewatering may be required during construction. The water removed would be settled and then discharged to a drainage way. A discharge permit would be obtained for these construction activities.

Construction-related traffic (e.g., materials delivery trips, workers, etc.) would access the site from Elverta Road. Materials trips would depend on geotechnical findings regarding the usability of the soil for foundations and the scheduling of construction activities. A traffic control plan would be prepared by the contractor and reviewed by Sacramento County to ensure traffic is safely routed past the work site. No off-site facilities are proposed for this project.

Safety on the construction site would be the responsibility of the contractor. The contractor would have a company safety program and a job-specific safety program, administered by a project safety officer. Typical procedures would include weekly safety meetings with the construction crew and hazard analyses prepared before the beginning of each new operation. OSHA and Cal-OSHA standards would apply for all work.

The construction contract documents would include a general SWPPP. The construction contractor would be required to submit a specific, more detailed SWPPP. The general plan would outline minimum requirements that must be met to minimize erosion and control sediments. The general and specific SWPPPs would comply with the county sediment and erosion control ordinances. Typical best management practices that would be used include the following:

- Covering all exposed slopes and stockpiles with plastic, straw, or hydroseed
- Placing silt fences at the downstream side of all work areas
- Placing a sediment filter in each drop inlet
- Sweeping all work areas frequently
- Constructing sediment ponds in key locations
- Placing waddles or hay bales across steep, disrupted slopes
- Constructing gravel driveways at the work site exit

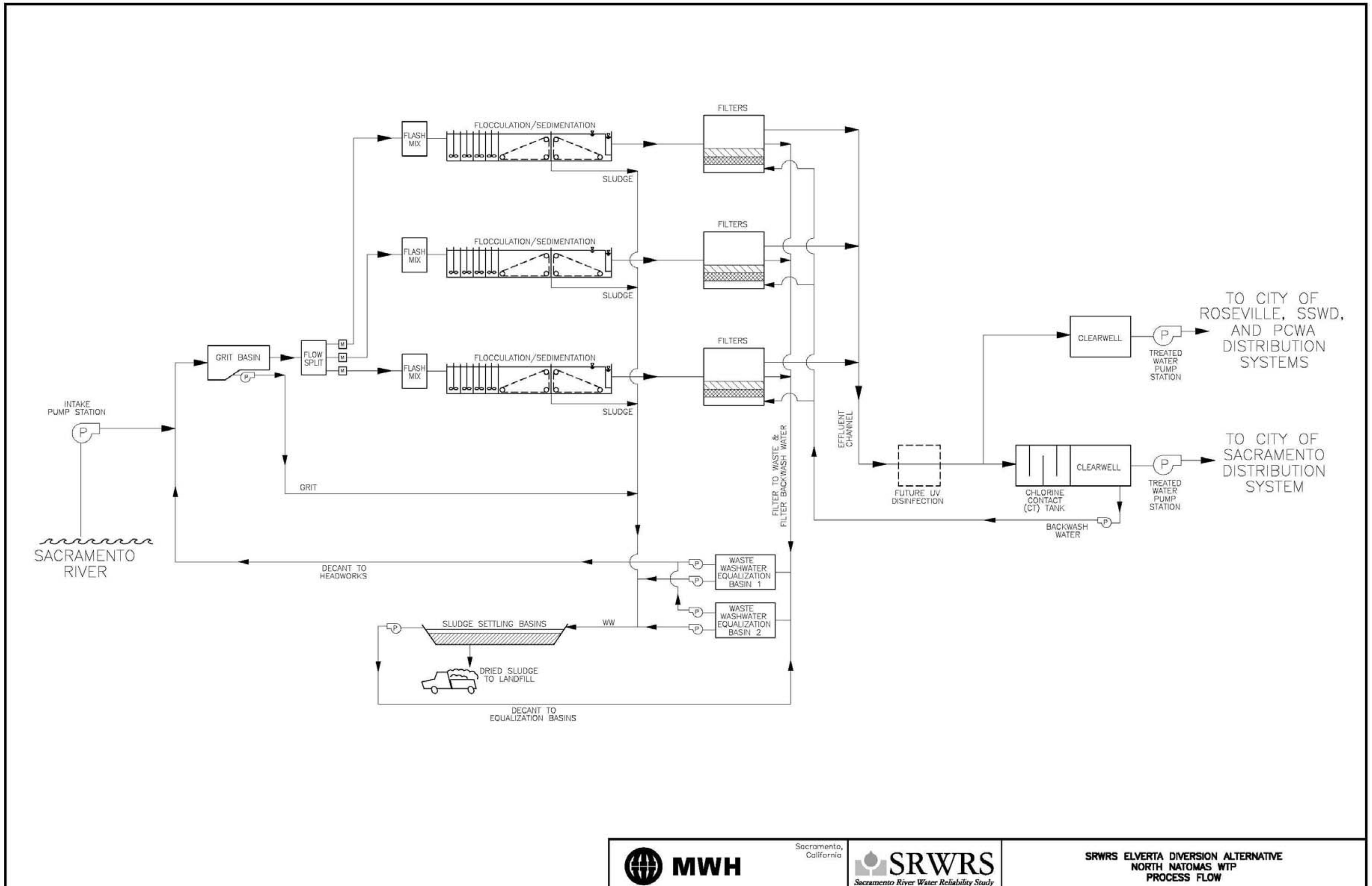

Figure 5-7 North Natomas WTP – Process Flow

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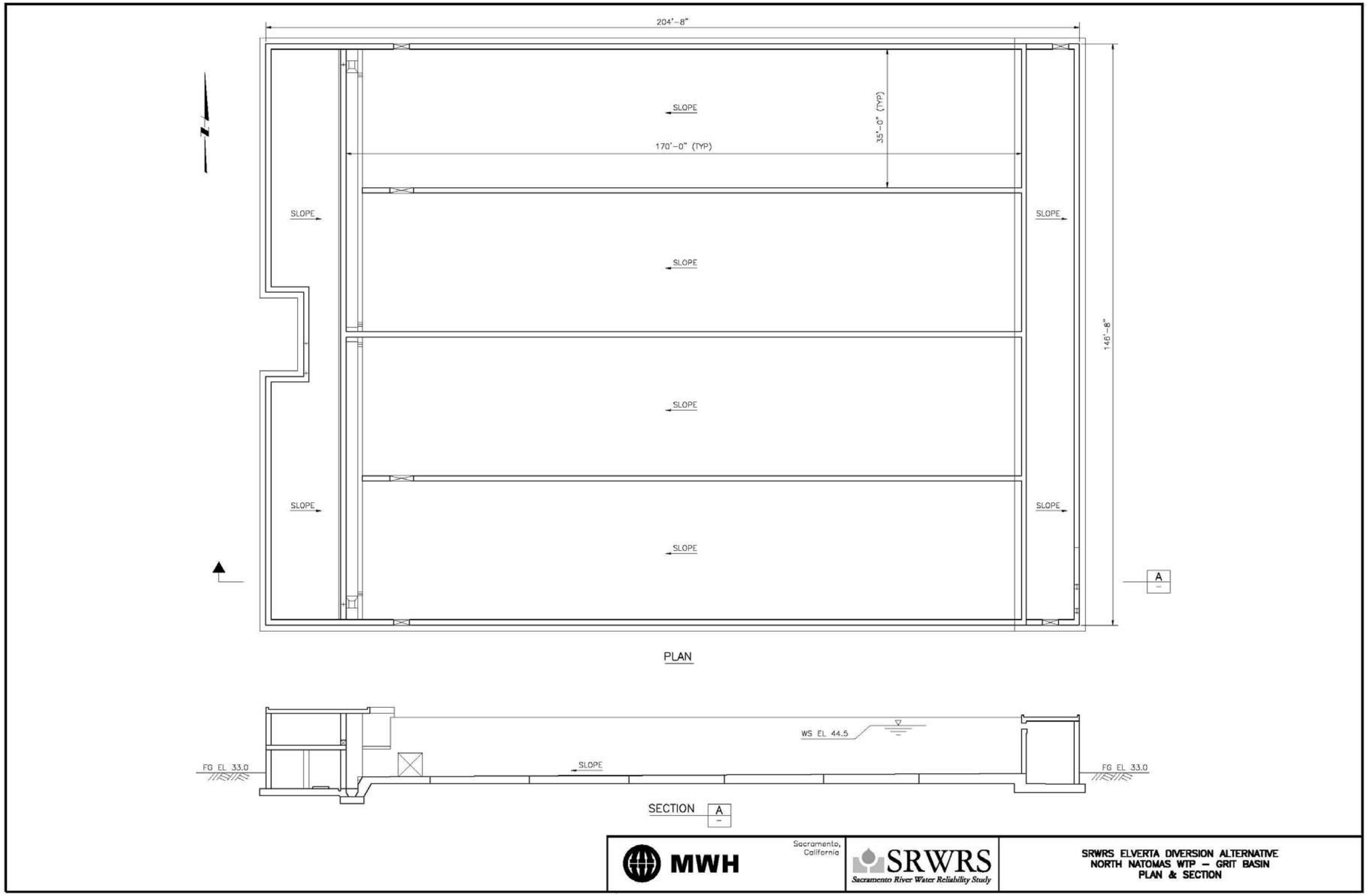

Figure 5-8 North Natomas WTP – Grit Basin Plan and Section

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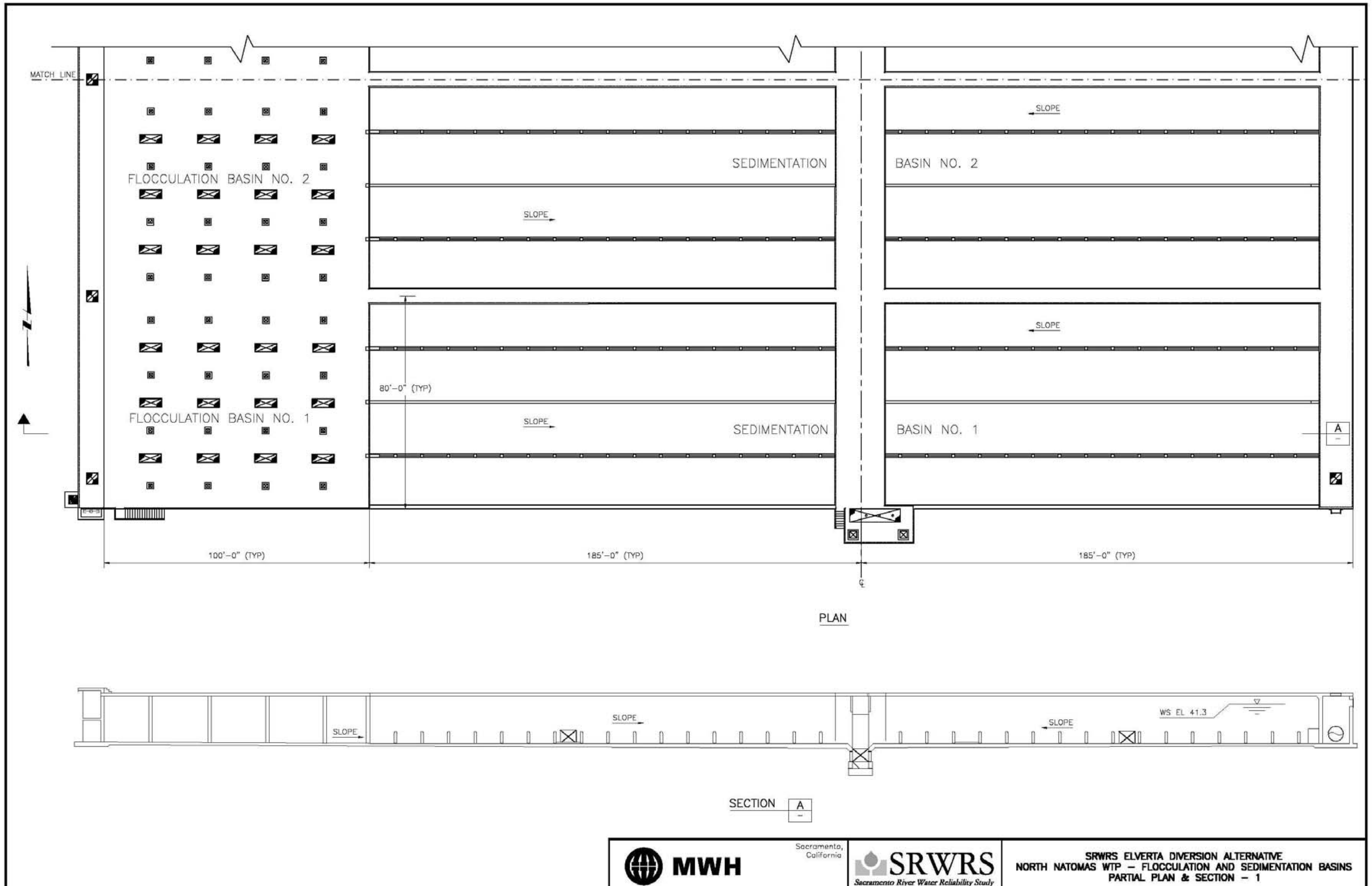

Figure 5-9 North Natomas WTP – Flocculation and Sedimentation Basins Partial Plan and Section-1

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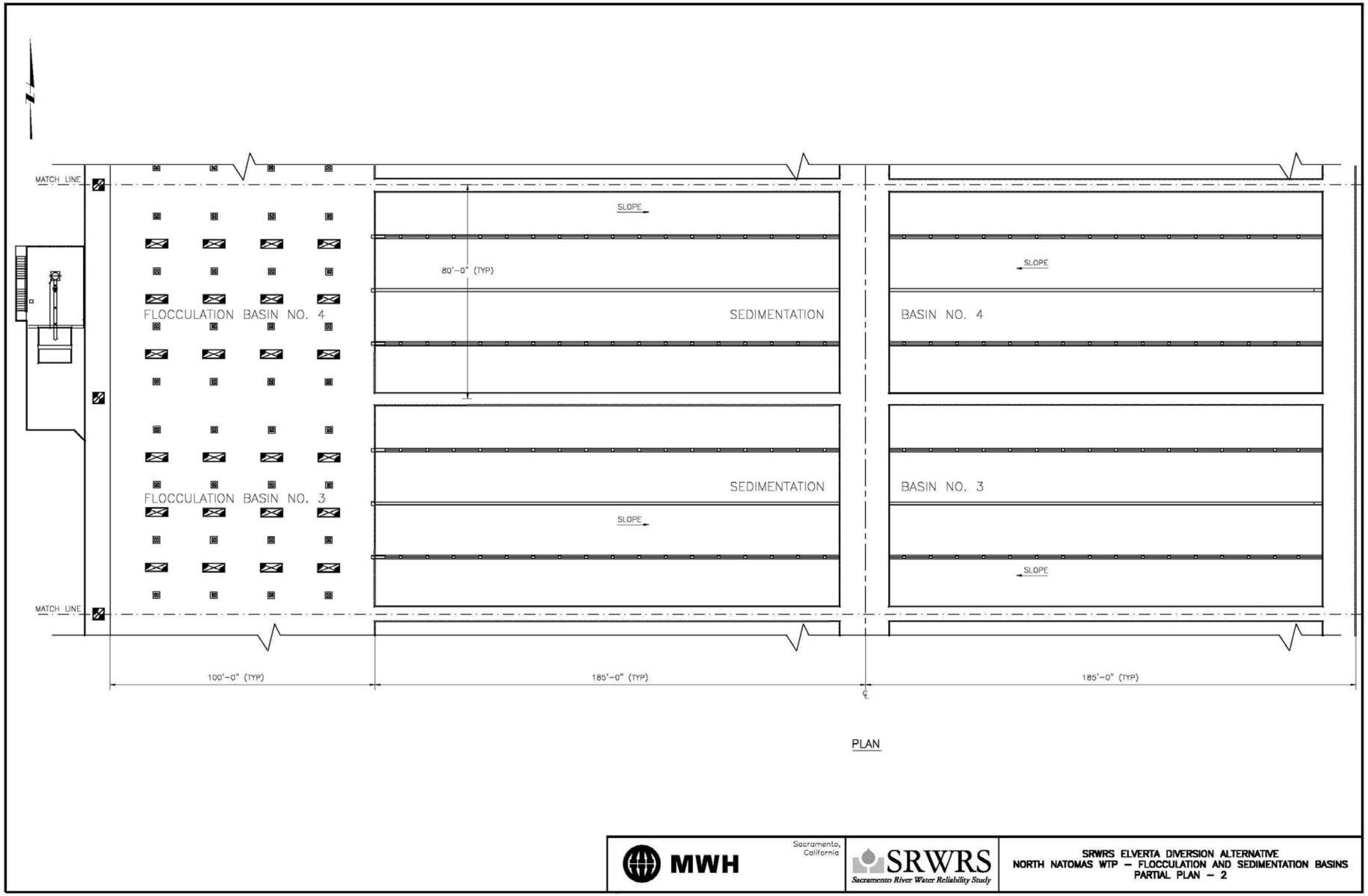

Figure 5-10 North Natomas WTP – Flocculation and Sedimentation Basins Partial Plan and Section-2

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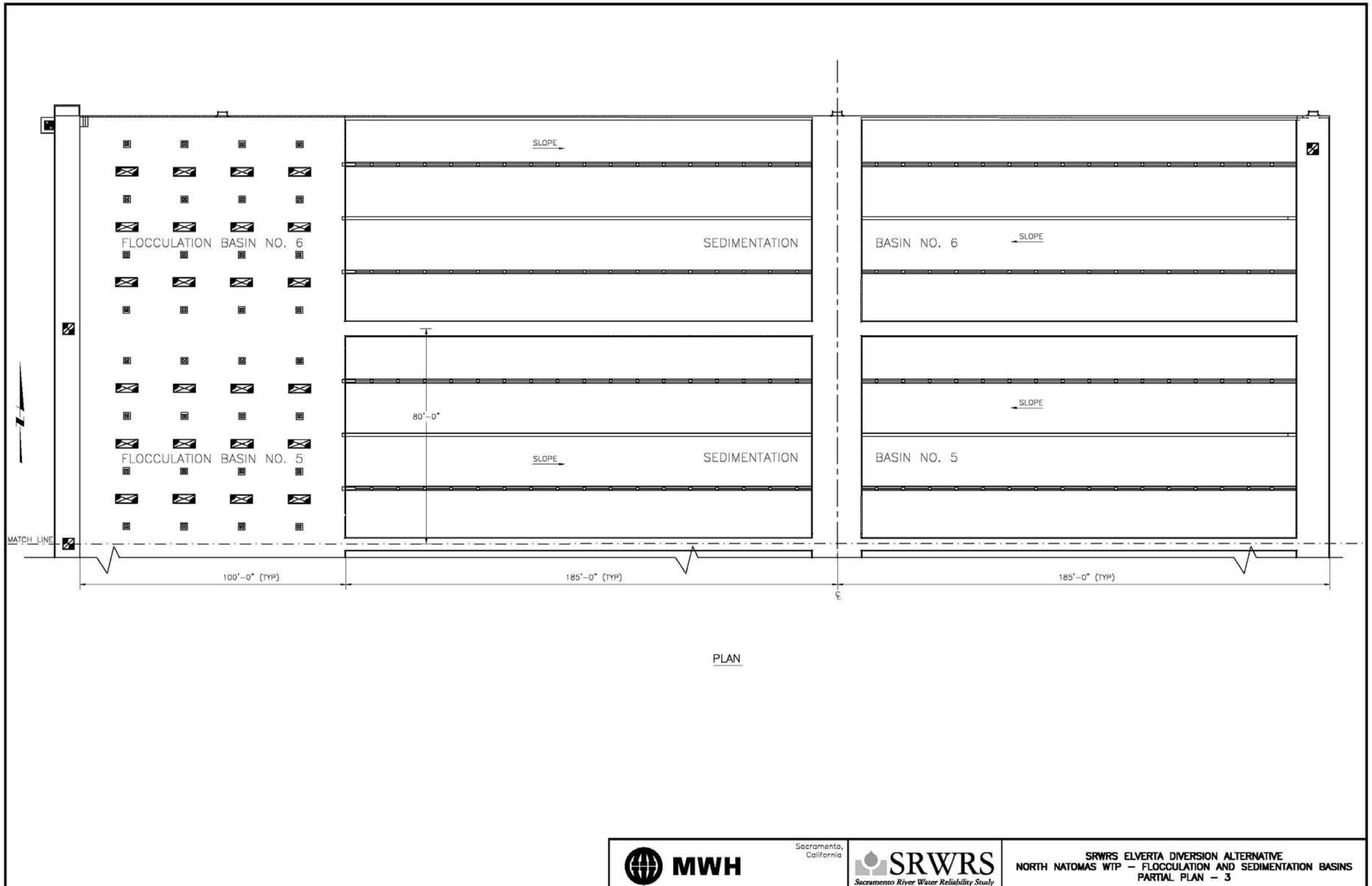

Figure 5-11 North Natomas WTP – Flocculation and Sedimentation Basins Partial Plan - 3

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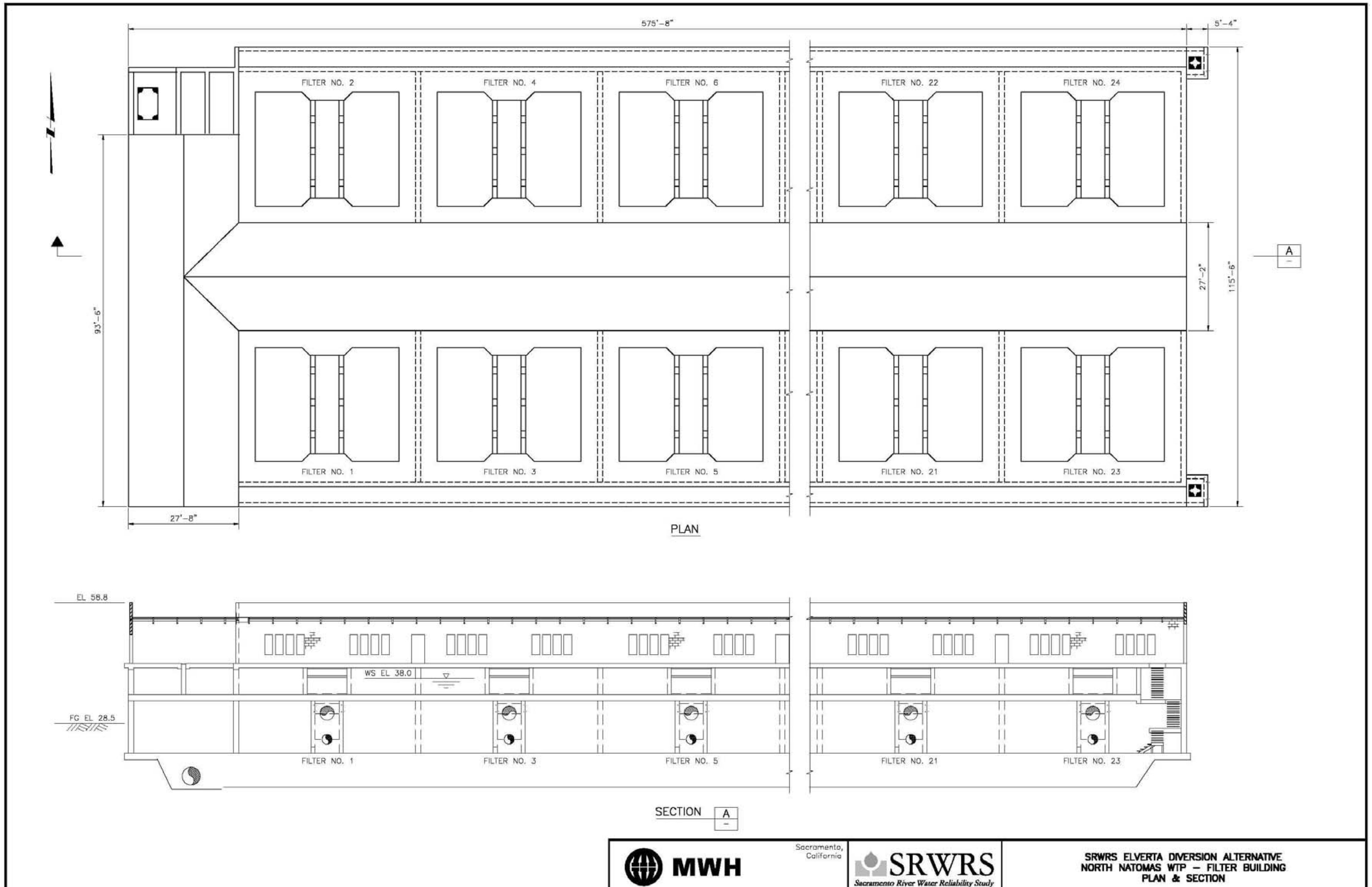

Figure 5-12 North Natomas WTP – Filter Building Plan and Section

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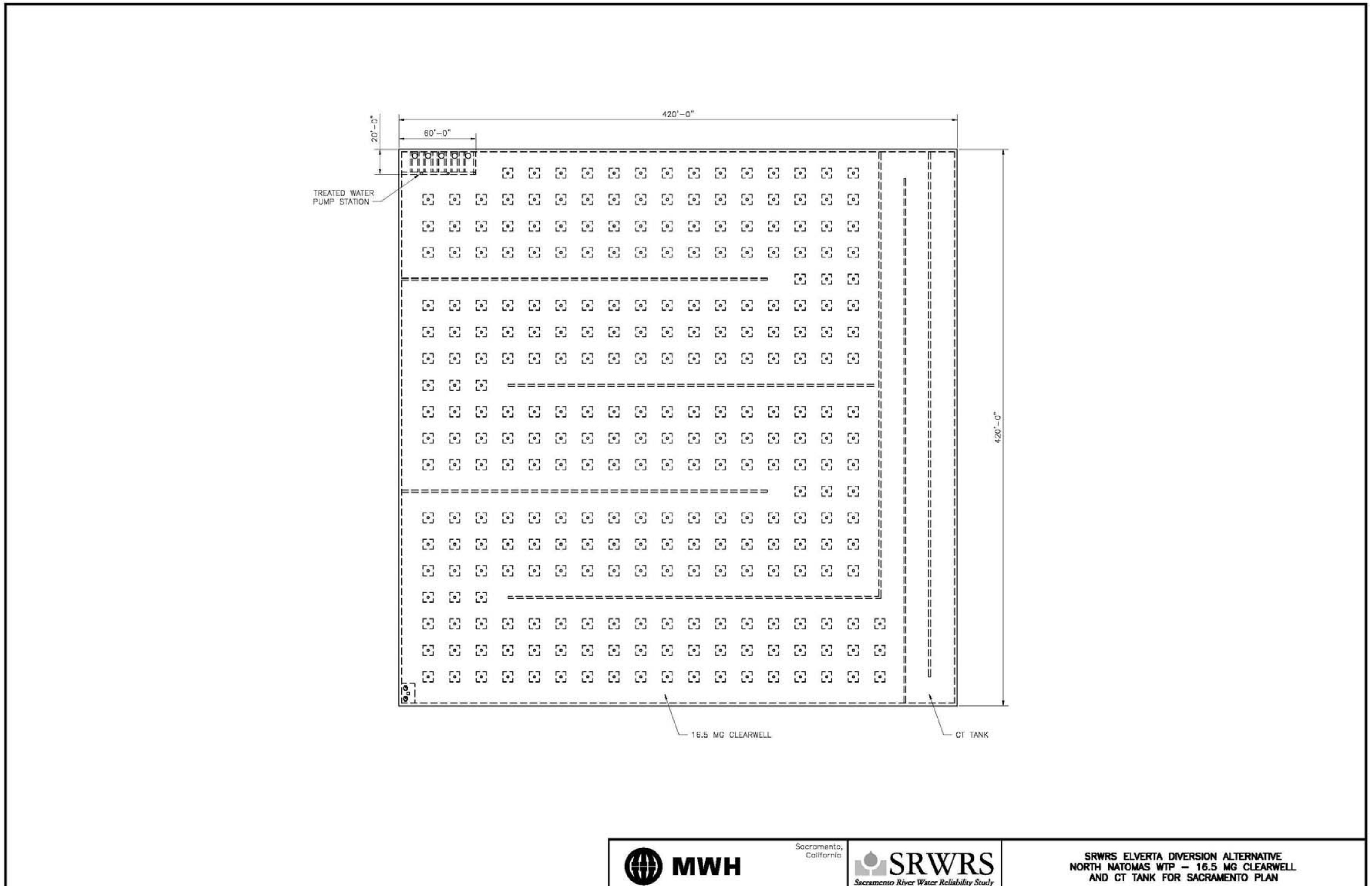

Figure 5-13 North Natomas WTP – 16.5 MG Clearwell and CT Tank for Sacramento Plan

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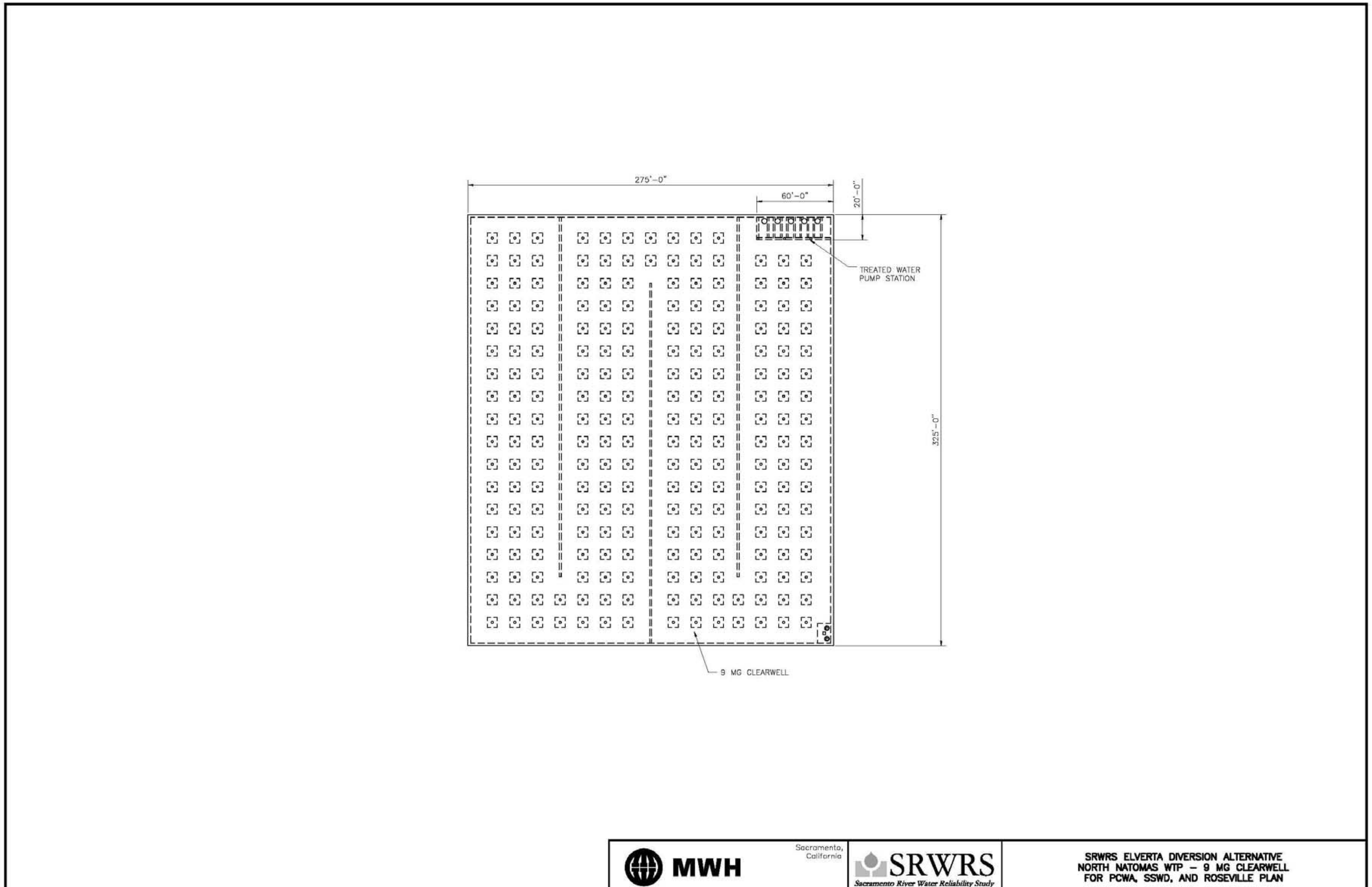

Figure 5-14 North Natomas WTP – 9 MG Clearwell Plan for PCWA, Roseville, and SSWD Plan

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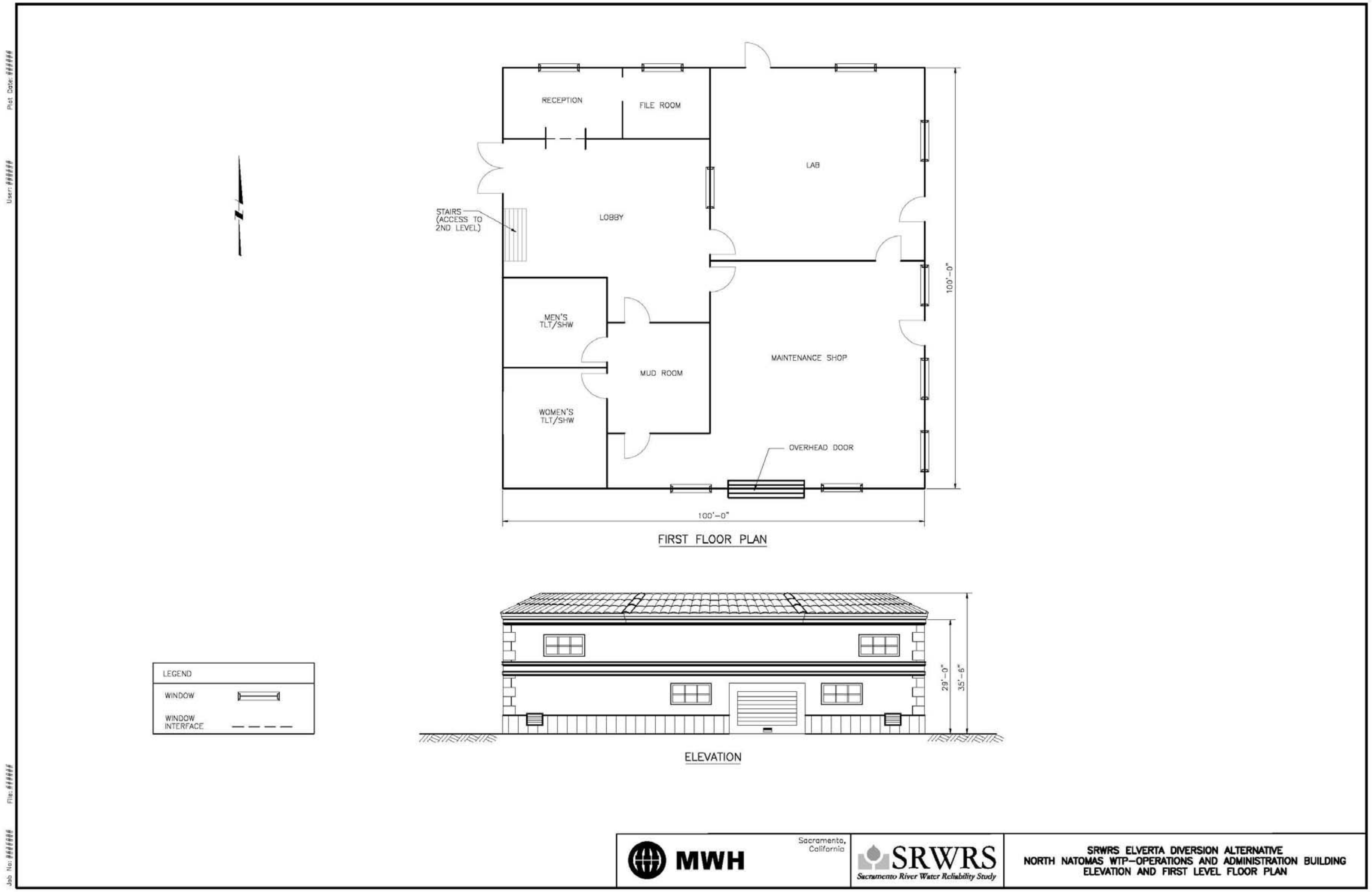

Figure 5-15 North Natomas WTP – Operations and Administration Building Elevation and First Level Floor Plan

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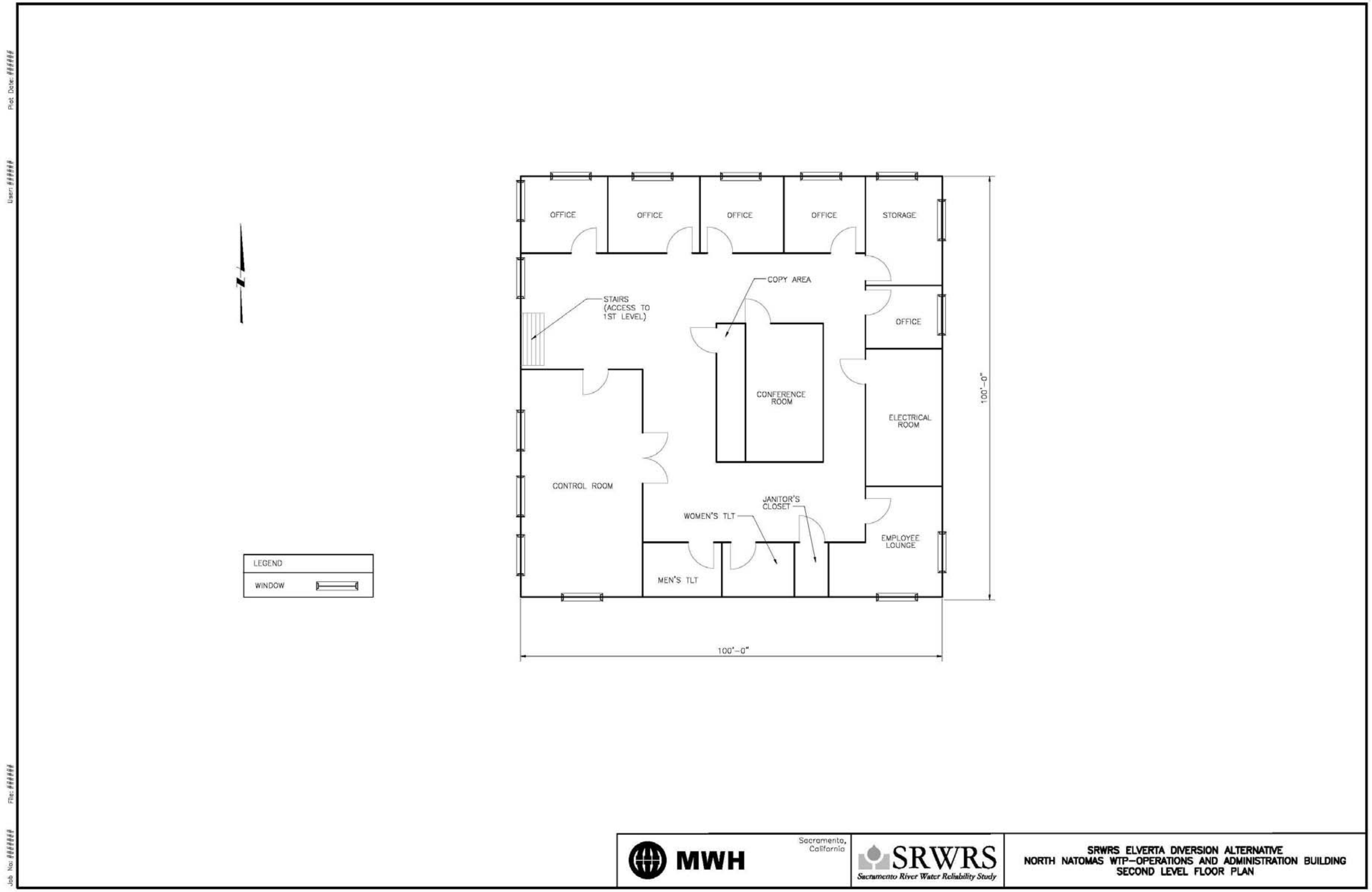

Figure 5-16 North Natomas WTP – Operations and Administration Building Second Level Floor Plan

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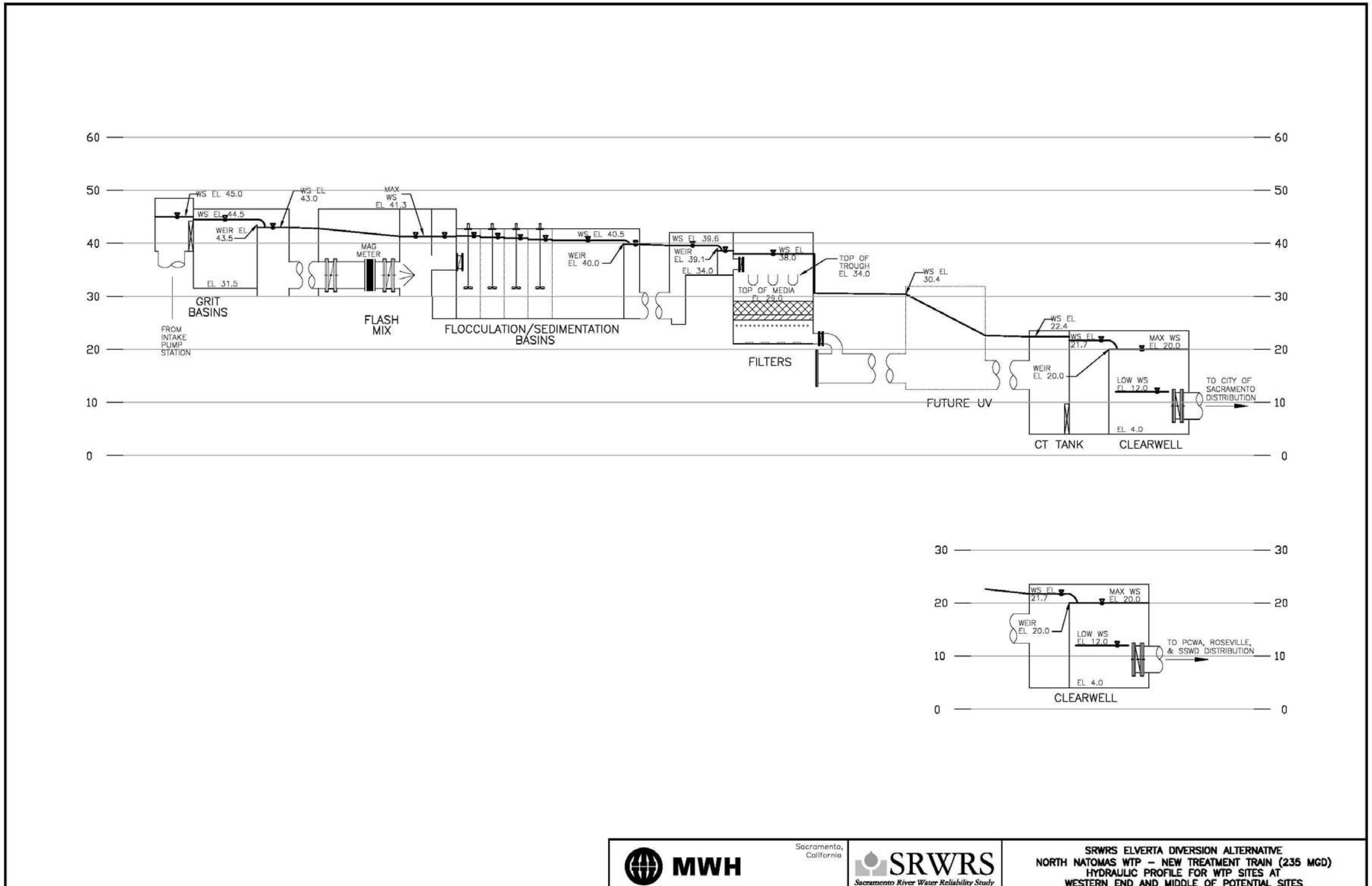

Figure 5-17 North Natomas WTP – New Treatment Train (235 MGD) Hydraulic Profile for WTP Sites Located at Western End and Middle of Potential Sites

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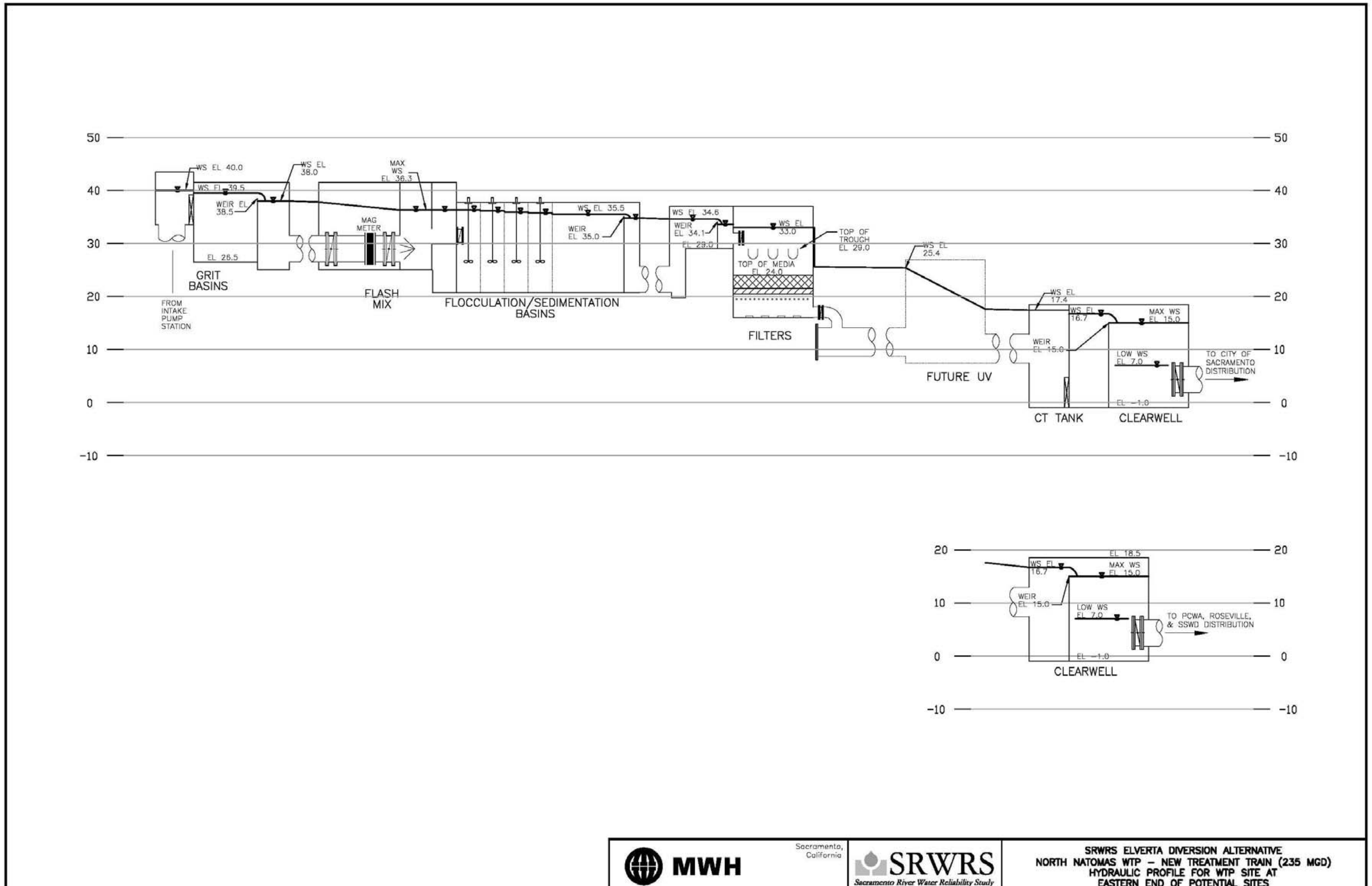

Figure 5-18 North Natomas WTP – New Treatment Train (235 MGD) Hydraulic Profile for WTP Sites Located at Eastern End of Potential Sites

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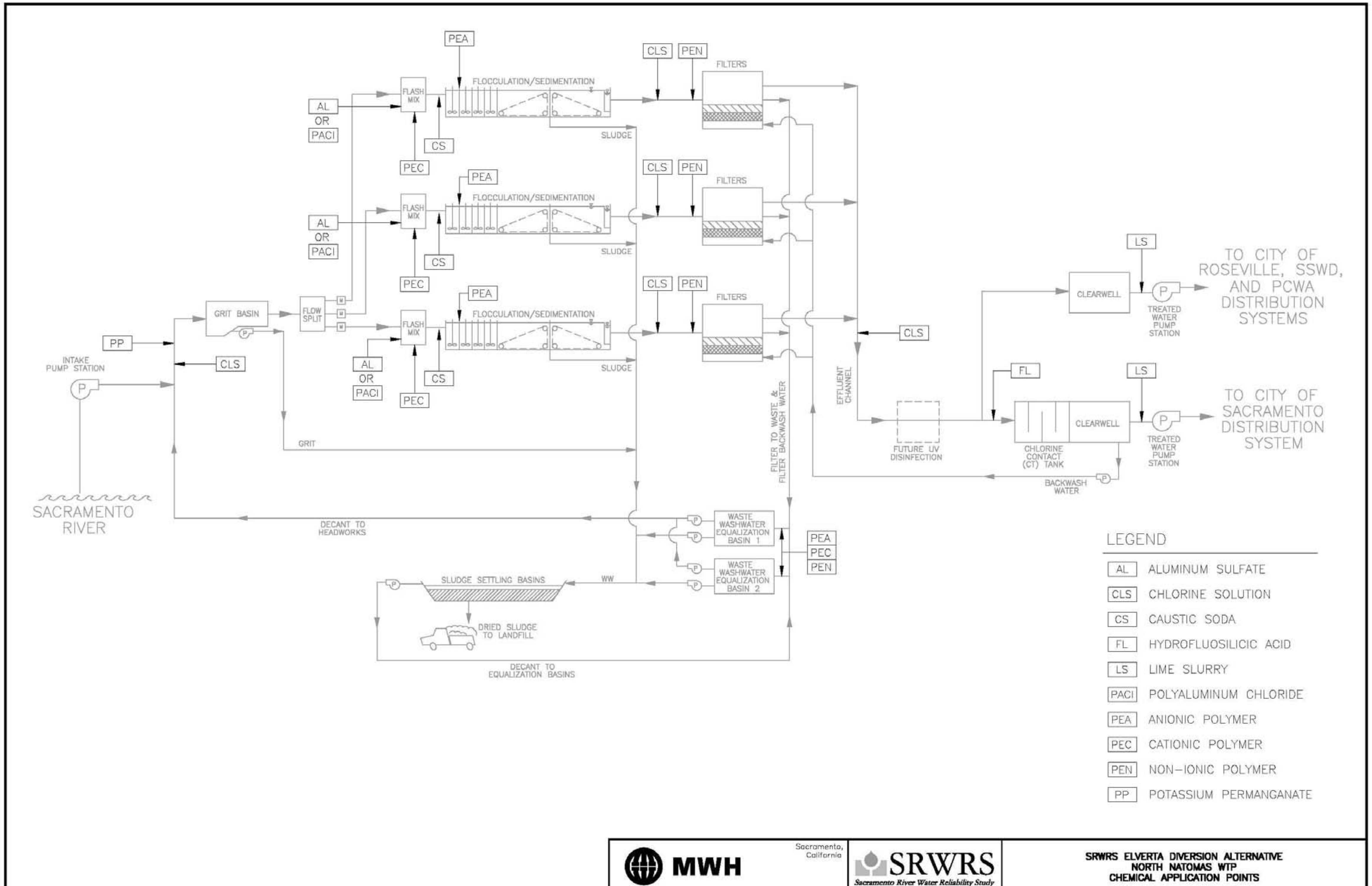

Figure 5-19 North Natomas WTP – Chemical Application Points

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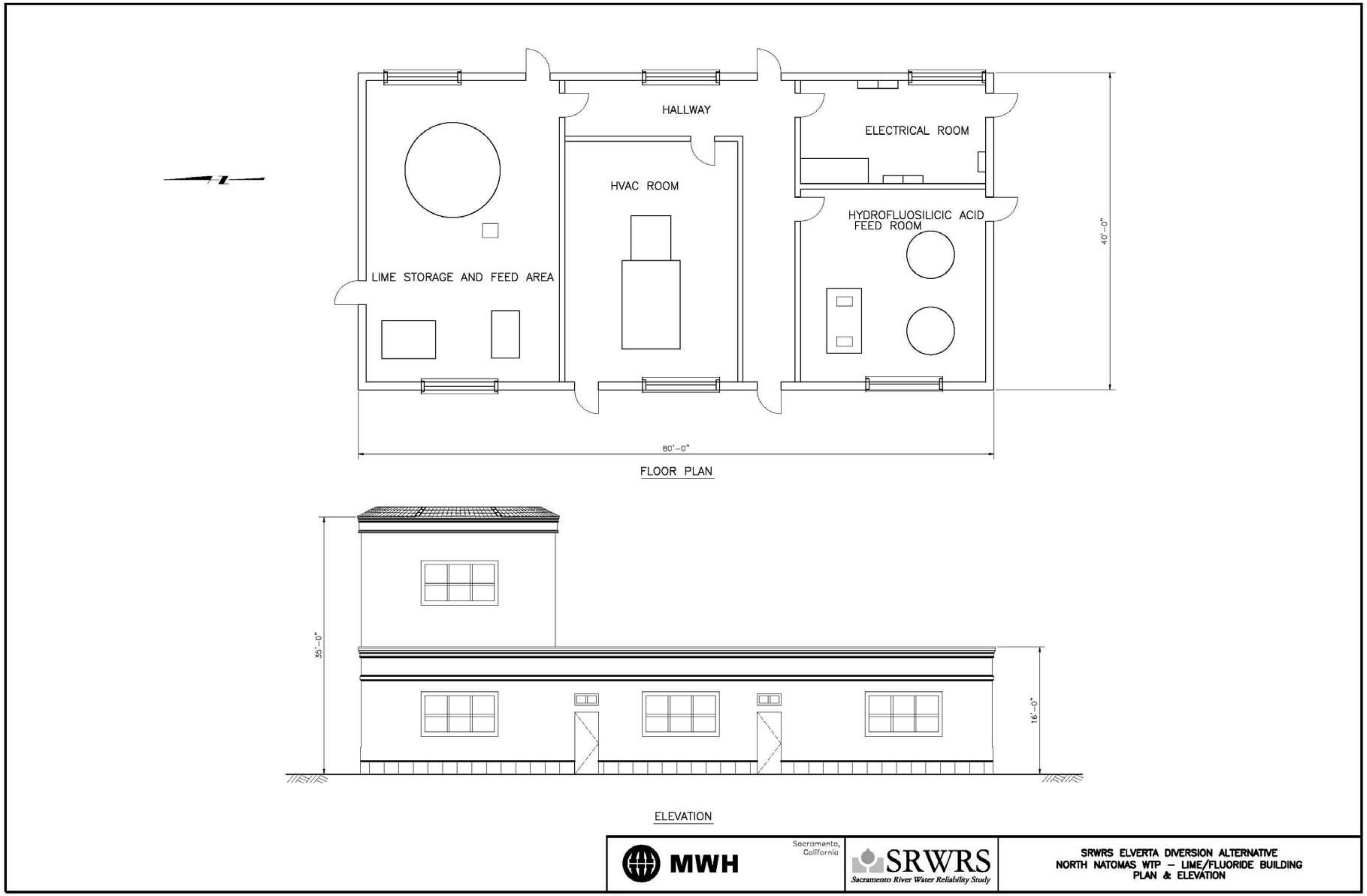

Figure 5-20 North Natomas WTP – Lime/Fluoride Building Plan and Elevation

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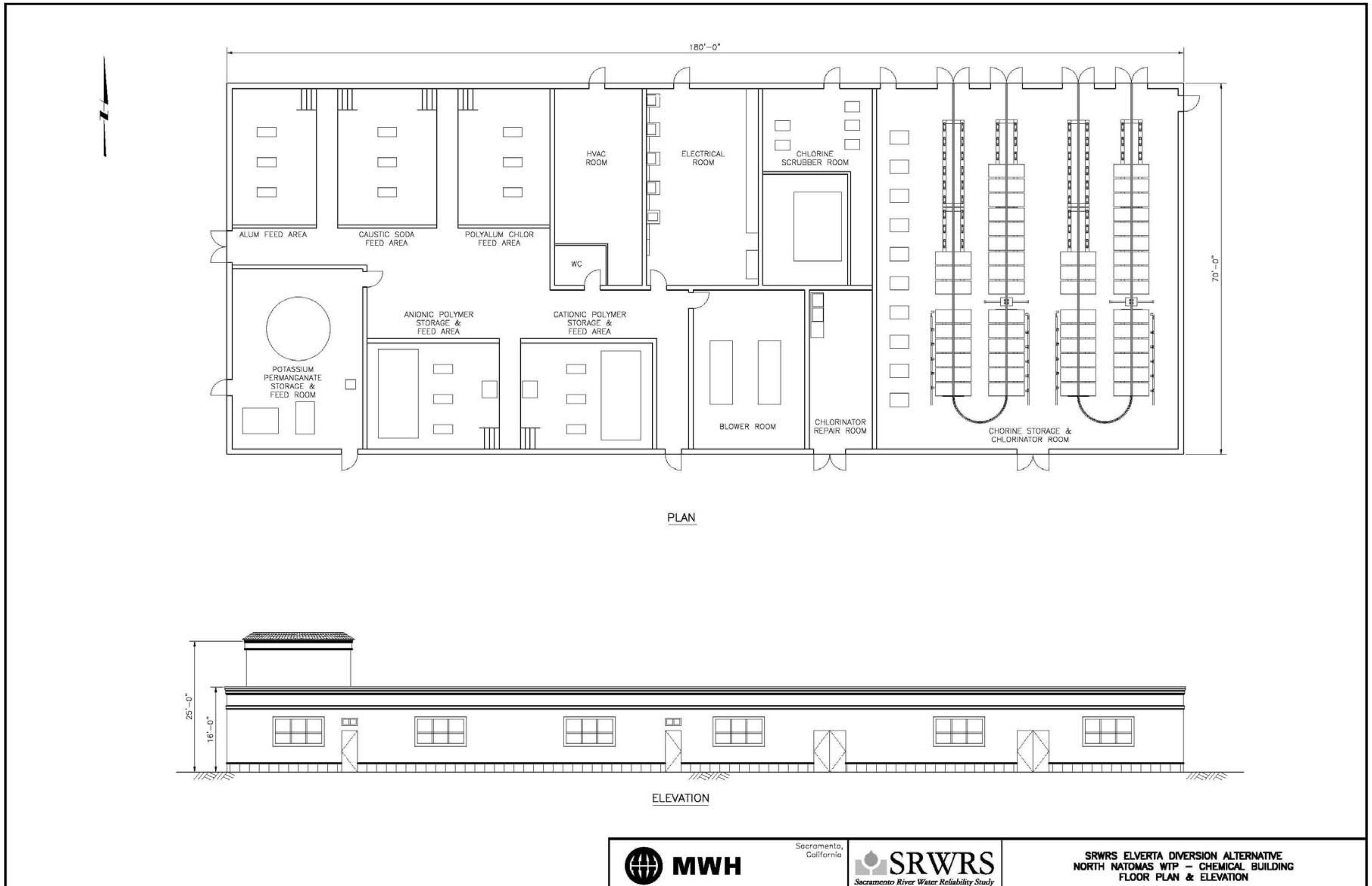

Figure 5-21 North Natomas WTP – Chemical Building Floor Plan and Elevation

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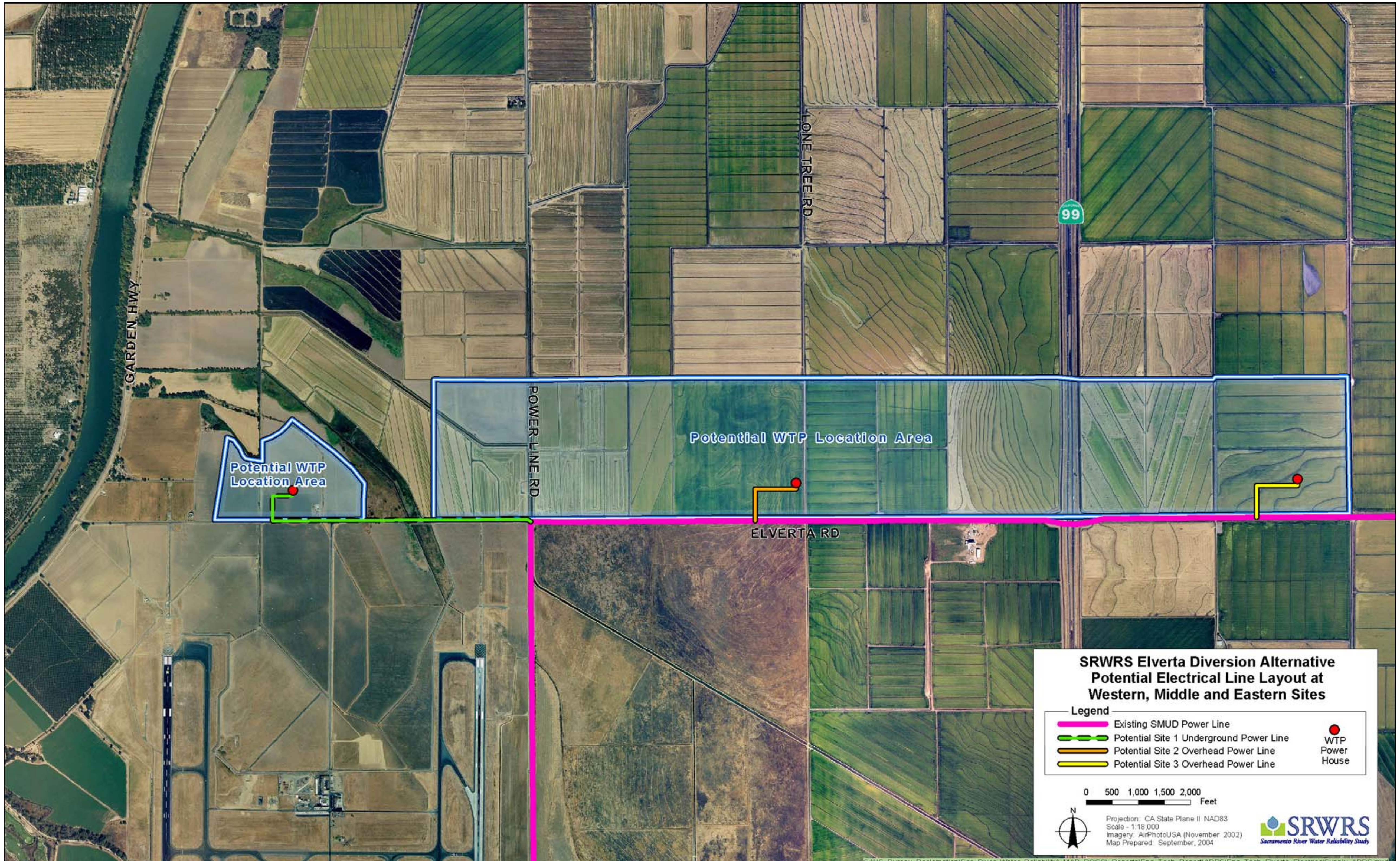

Figure 5-22 SRWRS Elverta Diversion Alternative Potential Electrical Line Layout at Western, Middle, and Eastern Sites

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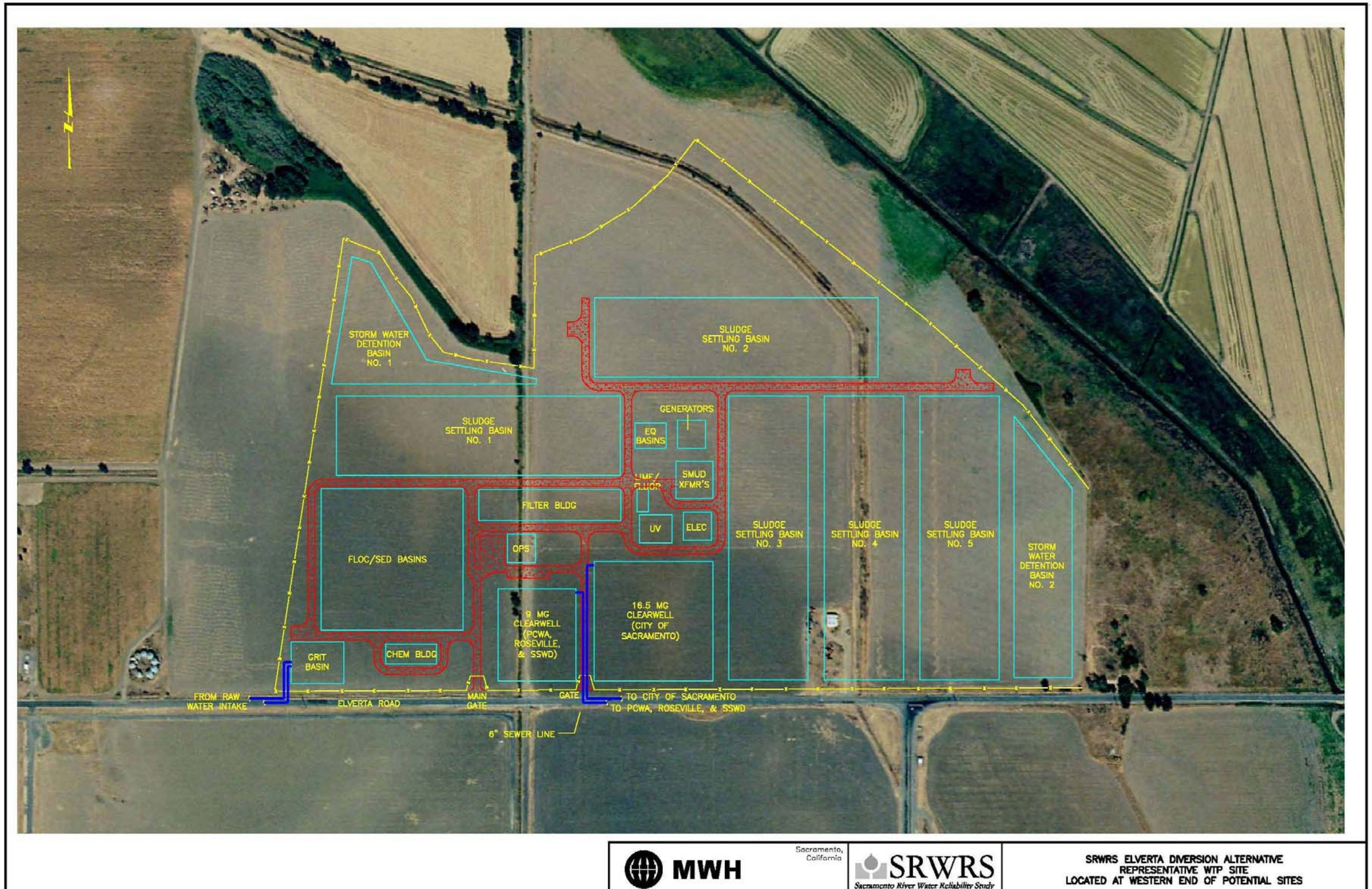

Figure 5-23 Representative WTP Site Located at Western End of Potential Sites

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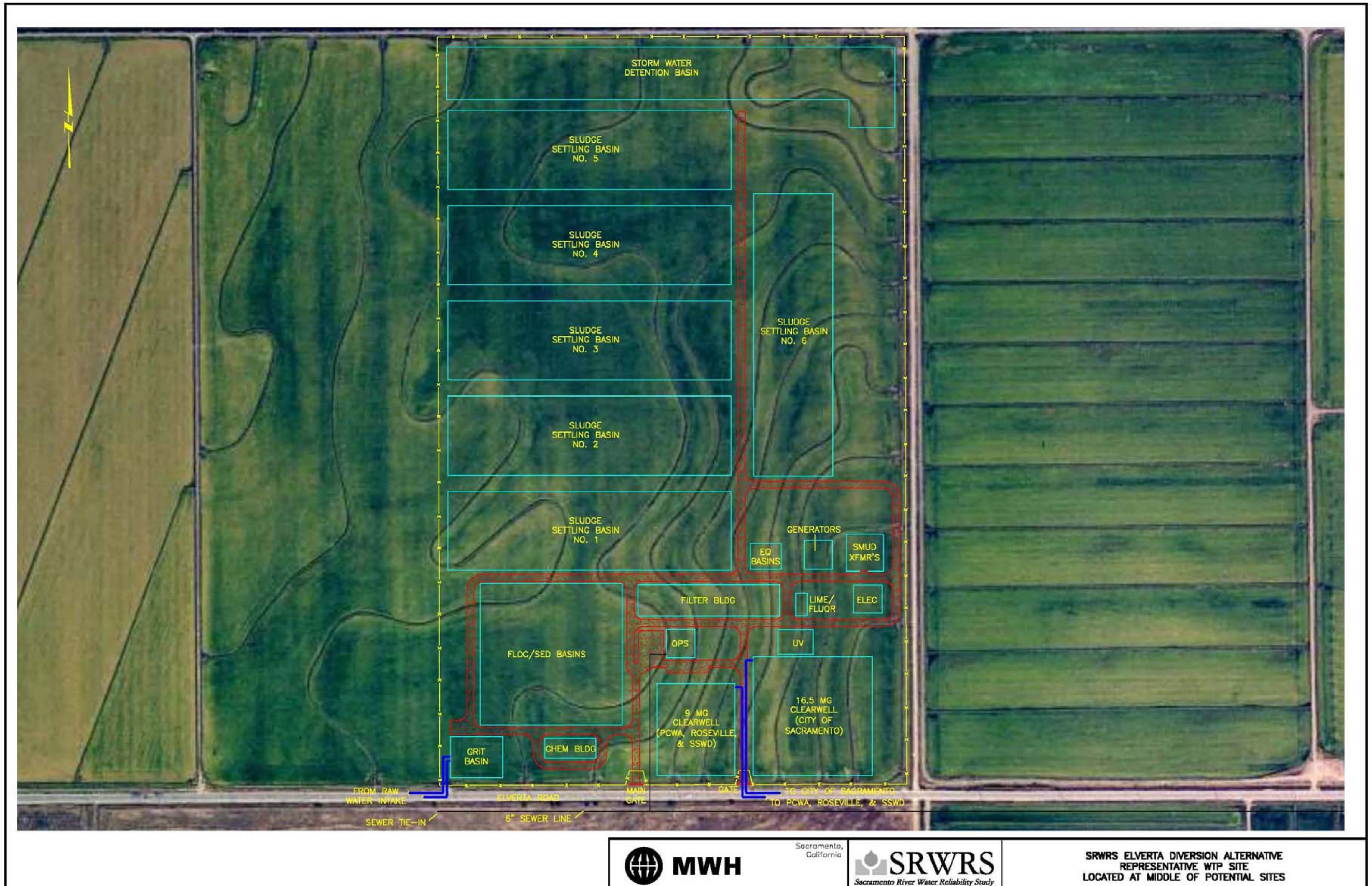

Figure 5-24 Representative WTP Site Located at Middle of Potential Sites

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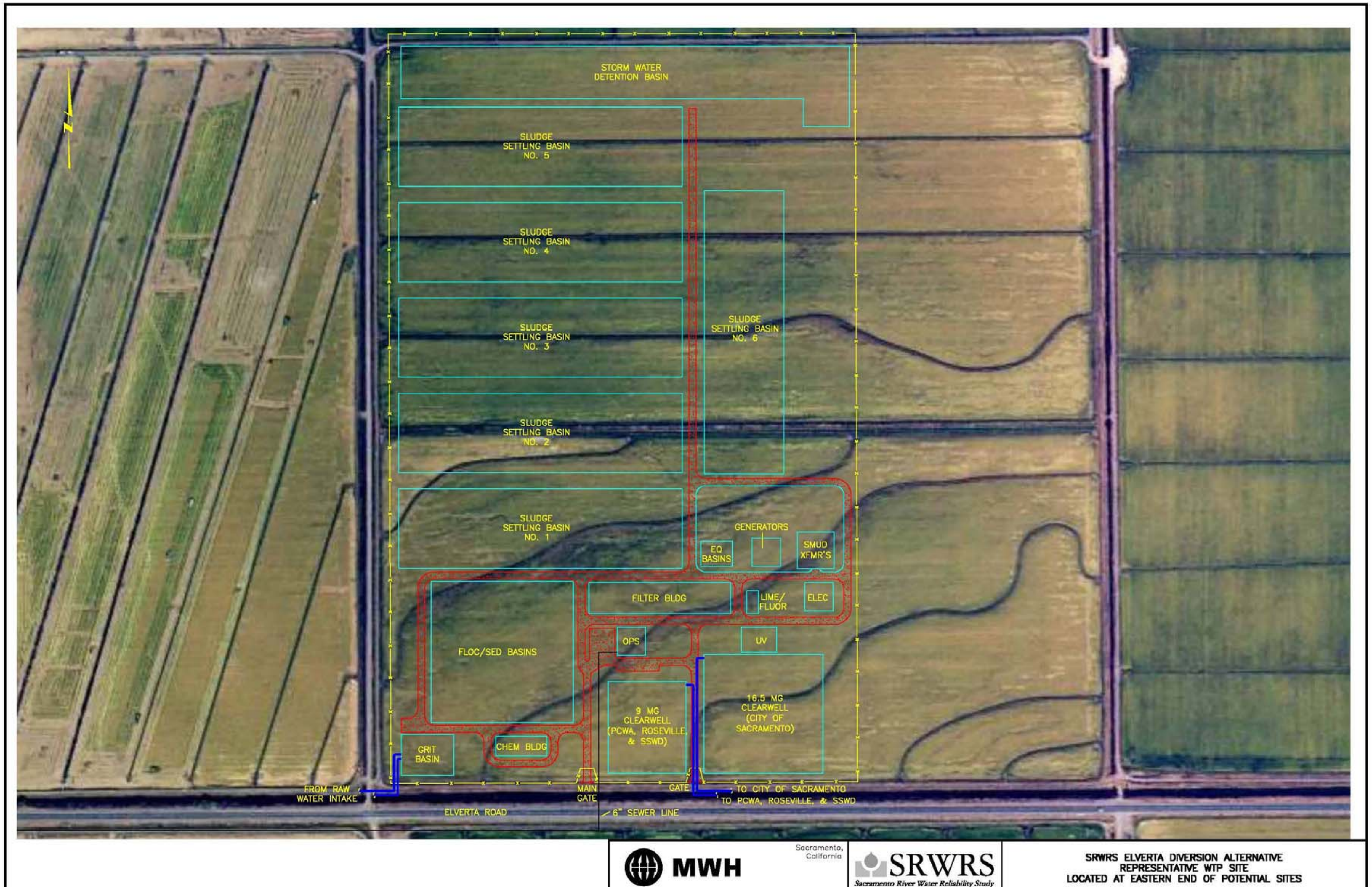

Figure 5-25 Representative WTP Site Located at Eastern End of Potential Sites

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CHAPTER 6 TREATED WATER PIPELINES

This chapter describes the pumps and pipes that would be used to convey water from the WTP to the four project partners. The systems discussed here are the major water transmission pumps and pipelines only. Storage of the treated water and distribution of that water to the ultimate users is the responsibility of each individual partner and is not part of this project. Treated water is to be conveyed to each of the four partners: PCWA, SSWD, Roseville, and Sacramento.

Two pump stations would be built at the WTP. The first pump station would pump into a transmission main that would deliver water to Sacramento. Multiple turnouts from this pipeline would connect it to Sacramento's distribution system. To maximize flexibility in water delivery, the pipeline has been sized so that all the water could be conveyed to the turnout at the intersection of the East Main Drainage Canal with Del Paso Road, the farthest point along the transmission main from the WTP.

The second pump station would pump into a transmission main that would deliver water to the other three partners. PCWA would receive water at three points: one along Baseline Road near Country Acres Lane, the second at the intersection of Baseline Road and Fiddyment Road, and the third at the intersection of Fiddyment Road and Athens Road. To maximize flexibility in water delivery, the pipeline has been sized so that all of PCWA's water could be conveyed to the turnout at Fiddyment Road and Athens Road, the turnout farthest from the WTP. SSWD would receive water through one turnout at the intersection of Walerga Road and Antelope Road. Roseville would receive water through a turnout along Fiddyment Road near Baseline Road.

6.1. HYDRAULICS

MWH used a water network model, H2ONet, to determine pipeline sizes, flow velocities, and heads. **Figure 6-1** presents the pipeline sizes recommended based on that computer analysis. **Figure 6-1** also presents the flow velocity in each pipe reach at peak flow. **Table 6-1** presents the design criteria used in the network analysis.

Four pump stations are required in the system. One of those pump stations is at the raw water intake and includes pumps for the SRWRS partners, and additionally pumps for NMWC in the Joint SRWRS-ARBFSHIP Elverta Diversion Alternative. The other three are in the treated water system. Two of the treated water pump stations are at the WTP with one pumping treated water to Sacramento and the other pumping treated water to the other three partners. The fourth pump station is a booster pump station just upstream from the Roseville turnout to boost water up to the higher pressure needed at the PCWA turnout at Athens Road and Fiddyment Road. **Table 6-2** presents basic statistics for each pump station.

Analysis were conducted analyses to test some of the assumed design criteria. Testing was performed to determine how sensitive the results were to the assumed pipe friction "C" value. If a "C" value of 120 (higher friction losses) is used instead of 140 the head losses in the system increase less than 6 percent. It was concluded that the system is not very sensitive to the assumed "C" values and that it is safe to remain with the initially assumed "C" value of 140 for all calculations.

The pipes shown in **Figure 6-1** were all sized to have a peak pipe velocity of about 5 fps. Testing was performed to determine how sensitive the system is to pipe velocity. Hydraulic analyses were conducted with pipe sizes that gave peak velocities of 8 fps and 10 fps, respectively. **Table 6-3** summarizes the total dynamic head that the Sacramento treated water pump station would have to pump under each of the three pipeline velocity scenarios.

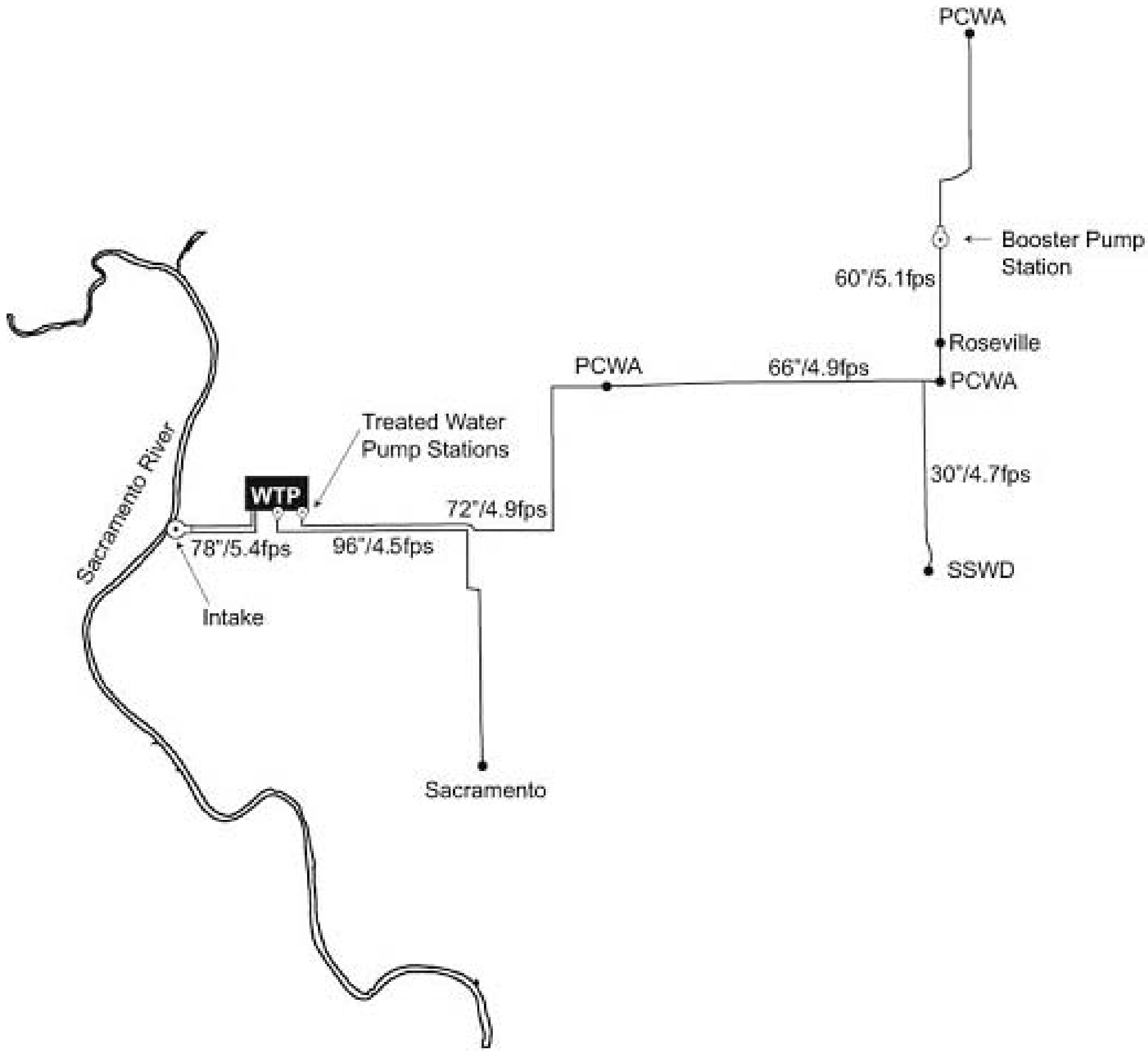

Figure 6-1 Pipe Size and Flow Velocities in Feet per Second at Design Flow for the SRWRS Elverta Diversion Alternative

Table 6-1 Design Criteria for Pipeline Hydraulic Analysis

Description	Planning Assumption
Turnout Flows	
City of Sacramento (one turnout at Del Paso Road)	145 mgd
PCWA Placer Vineyards Turnout at Baseline Road near Country Acres Lane	22.6 mgd
PCWA Dry Creek-West Placer Turnout at Baseline Road near Fiddyment Road	3.6 mgd
PCWA Turnout at Fiddyment Road and Athens Avenue (size the pipelines to convey the full PCWA flow of 65 mgd to this point)	38.8 mgd
Roseville (one turnout on Fiddyment Road near Baseline Road)	10 mgd
SSWD Turnout (one turnout near Antelope and Walerga roads)	15 mgd
Turnout Delivery Pressure	
City of Sacramento	50 psi
PCWA Placer Vineyards Turnout at Baseline Road near Country Acres Lane	290 feet above msl
PCWA Dry Creek-West Placer Turnout at Baseline Road near Fiddyment Road	290 feet above msl
PCWA Turnout at Fiddyment Road and Athens Avenue	350 feet above msl
Roseville	290 feet above msl
SSWD Turnout	280 feet above msl
Storage Reservoirs	
Locate storage at the WTP equal to the operational storage requirements (i.e., capacity to handle differences between plant flow and pumped flow, estimated as 0.1 x maximum daily flow). No other storage to be provided by this system.	25 MG
Miscellaneous Modeling Criteria	
Pipe Velocity	5 fps
Pipe Friction Hazen Wouldiams factor	140
Pipe Incidental Losses (fittings per 1,000 feet of pipe)	6

Key:
fps – feet per second
MG – million gallons
mgd – million gallons per day
msl – mean sea level
PCWA – Placer County Water Agency
psi – pounds per square inch
SSWD – Sacramento Suburban Water District
WTP – water treatment plant

Table 6-2 Pump Station Hydraulic Characteristics

Pump Station	Peak Flow (mgd)	Discharge W. S. El. (ft)	Supply W.S. El. (ft)	Friction Head Loss (ft)	TDH (feet)	Pump Efficiency	Connected Horsepower(1)
Intake	235	45	4	12.5	53.5	70%	3,600
Intake for Natomas	136	40	4	1	37	70%	1,600
City of Sacramento	145	130	15	46	161	80%	6,400
PCWA/Roseville/SSWD	90	290	15	144	419	80%	9,950
PCWA Booster	38.8	350	290	18	78	80%	885

Note:
(1)Connected horsepower includes an allowance for a backup pump.
Key:
ft – feet
mgd – million gallons per day
PCWA – Placer County Water Agency
SSWD – Sacramento Suburban Water District
TDH – total dynamic head
W.S. El. – water surface elevation

As shown in **Table 6-3**, head loss in the system is quite sensitive to pipe velocity. Maintaining design pipe velocities at 5 fps is recommended.

Table 6-3 Effects of Pipeline Velocity on Pump Station Dynamic Head

Pipe Size and Peak Flow Velocity	Pump Station Total Dynamic Head (ft)	Pump Station Connected Horsepower
Velocity = 5 ft/s; Pipe Diameter = 96"	183	7,007
Velocity = 8 ft/s; Pipe Diameter = 78"	270	11,750
Velocity = 10 ft/s; Pipe Diameter = 72"	331	14,405

Note:
Flow rate = 165 mgd
Key:
ft – feet
ft/s – feet per second

6.2. PREFERRED ALIGNMENT

The selected treated water pipeline alignments are presented in **Figure 1-1**. More detailed plans and profiles for the pipeline are presented in **Figures 6-2** through **6-9** (at end of chapter**)**. The treated water pipeline feeding Sacramento would leave the pump station at the WTP and follow Elverta Road east to the East Main Drainage Canal where it turns south. The pipeline would be placed outside the levee on the east side of the East Main Drainage Canal. The pipeline would end at Del Paso Road where it would connect to the Sacramento water distribution system. Along Elverta Road, the pipe would be constructed approximately at the fog line (the white line on the edge of the outside lane of the road) on the north side of the road, although it would shift north or south occasionally to avoid obstacles (structural or environmental).

The treated water pipeline for PCWA, SSWD, and Roseville would originate at a pump station at the WTP and follow Elverta Road east to Sorento Road. The pipeline would follow Sorento Road north to the Sacramento/Placer County line where the road name changes to Pleasant Grove Road. The pipeline would follow Pleasant Grove Road north to Baseline Road. The pipeline would turn east and follow Baseline Road. One or more turnouts along this pipe would feed PCWA distribution systems along Baseline Road.

A "tee" would occur in the pipeline at the point where the old Walerga Road intersected with Baseline Road. One leg of the tee would continue along Baseline Road. The other leg from this tee would follow the abandoned section of Walerga Road and then Walerga Road south to Antelope Road where it would connect to the SSWD distribution system.

The pipeline along Baseline Road would continue to the intersection with Fiddyment Road. A turnout would occur for service to PCWA at Fiddyment Road and Baseline Road. The pipe would then turn north on Fiddyment Road. The one turnout for service to Roseville would be along Fiddyment Road just north of Baseline Road. The pipeline would continue north on Fiddyment Road to Athens Road where another turnout would occur for service to PCWA.

Design work is currently underway for a realignment of Fiddyment road north of Blue Oaks Boulevard. The pipeline would follow this realigned Fiddyment Road. The exact location of the pipeline within this new section of Fiddyment Road would be determined during final design of that road section.

Plans call for the rest of Fiddyment Road to be widened. This pipeline would be placed under the existing Fiddyment Road pavement between Pleasant Grove Road and Blue Oaks Boulevard, which would become the northbound side of the widened roadway. Between Baseline Road and Pleasant Grove Road, the pipe would be installed outside the western edge of the existing pavement.

6.3. ALIGNMENT EVALUATION AND SELECTION

The preferred alignment was selected after consideration of several alternative alignments. One set of alternatives involved routing the PCWA/SSWD/Roseville pipeline along Elverta Road instead of along Baseline Road. Baseline Road was selected because it involved one crossing of Dry Creek as opposed to two Dry Creek crossings for the Elverta Road route, and the Baseline Road route was through a less developed area meaning less disruption of residences. A technical memorandum describing in more detail the comparison of these two alternatives is included in **Appendix C**.

Another alternative considered involved routing the pipe from Baseline Road to Athens Road along the roads to be constructed in the West Roseville area instead of along Fiddyment Road. The alternative alignment would have included a stretch of pipeline constructed in open country between Baseline Road and the southern edge of the West Roseville area. This open area includes a number of wetlands. Timing the construction of the pipeline in the West Roseville area would have been difficult. If the pipeline were constructed before the roads, it would be through open country and would present a large number of environmental impacts. If the pipeline were constructed after the roads were built, its construction would damage the new roads. It is unlikely that the SRWRS project would be ready for construction at the same time as the new streets. The route along Fiddyment Road was selected because it avoided these environmental and scheduling complications.

Another set of alternatives considered involved the route for the pipe connecting to the SSWD system. In addition to the recommended route, an alternative of going south on Watt Avenue from Baseline Road to Antelope Road and east on Antelope Road to Walerga Road was considered. Analysis showed that the Watt Avenue route would have been more costly, primarily because it involved a greater length of larger pipe. The Walerga Road alternative also affords the opportunity of hanging the pipe from the new bridge over Dry Creek that is being planned for Walerga Road. Hanging the pipe from the new bridge would be cheaper than tunneling under Dry Creek. It is unlikely that the old bridge over Dry Creek along Watt Avenue would be able to take the load of a pipe, but the new bridge along Walerga can be designed with this pipe load in mind. A technical memorandum describing in more detail the comparison of these two alternatives is included in **Appendix D**.

6.4. TUNNEL SECTIONS AND OTHER SPECIAL CROSSINGS

The treated water pipeline would cross several drainage ways where the pipe may need to be constructed using trenchless technologies to protect wetland habitat. At this time, the locations listed in **Table 6-4** have been identified as places the pipeline might need to be tunneled under a creek or other drainage way. It is possible that biological surveys yet to be conducted would identify additional locations where trenchless technology might be the preferred construction method.

Table 6-4 Locations Where Trenchless Technology May Be Used for Pipeline Construction

Description of Location	Approximate Crossing Length (feet)	Approximate Pipeline Station
Jacobs Slough (two parallel pipes)	100	70+25
Highway 99 at Elverta Road (two parallel pipes)	300	190+85
East Main Drainage Canal at Elverta Road (two parallel pipes)	200	270+90
Natomas East Main Drainage Canal at Elverta Road	300	350+00
Dry Creek at Walerga Road (tunnel or in bridge approach and hung from new bridge)	400	880+75
Pleasant Grove Creek in West Roseville	300	1010+25

Trenchless technology (tunneling) involves digging a jacking pit on one side of the drainage way and a receiving pit on the other side, then using a tunneling machine to bore between the two pits and a jacking machine to push pipe through the hole. Another trenchless technology, known as directional drilling, may be preferred for pipes less than about 48 inches in diameter. If directional drilling is used, the jacking and receiving pits are eliminated and the pipe, when installed, forms an inverted arc under the drainage way.

Placer County is planning to improve the Walerga Road bridge over Dry Creek. It may be possible to design the bridge so that the transmission main for this project can be hung from the bridge. This would be a less expensive way to construct the pipe across Dry Creek than using tunneling or directional drilling to install the pipe under the creek.

6.5. PCWA BOOSTER PUMP STATION

The PCWA turnout at Fiddyment Road and Athens Road requires a delivery head of 350 feet msl while the turnout for the Roseville requires a head of 290 feet msl. A booster pump station is needed to boost the pressure in the transmission main downstream of the Roseville turnout where the pipeline enters PCWA's Zone 1 service area.

The booster pump station would pump a peak flow of 38.8 mgd at a maximum total dynamic head of 78 feet. The connected horsepower at the station would be 885 hp, including one spare pump. It is expected three vertical turbine pumps would be installed on a slab at the site. The pump slab would measure about 30 feet by 20 feet.

In addition to the pumps themselves there would be an electrical and control building for the booster pump station. This building would house the electrical switchgear, controls, and telemetry equipment for the pump station. The electrical and control building would be approximately 500 square feet in size.

The maximum power requirement for the 38.8 mgd Booster Pump Station has been estimated to be 887 kilovolt-amperes (kVA). **Table 6-5** summarizes how the general power requirements were estimated.

Table 6-5 Power Requirement Summary for the 38.8 mgd Booster Pump Station

Pump Station	Peak Flow (mgd)	Pump Load[(1)] (hp)	Misc. Loads (kVA)	Power (kVA)	Amps @ 480 Volts	1/2 Load (kVA)
PCWA Booster Pump Station	38.8	885	2	887	1,067	443

Note:
(1) Includes a spare pump.
Key:
hp – horsepower
kVA – kilovolt-ampere
mgd – million gallons per day
PCWA – Placer County Water Agency

The proposed primary backup power supply option is the use of a diesel generator at the Booster Pump Station site. The SRWRS partners selected a 50 percent backup generation capacity for evaluation. The required 50 percent backup generation for the 38.8 mgd Booster Pump Station would require a 450 kVA generator and a fuel storage tank. The space required for this equipment is approximately 1,000 square feet. The generator could be located in a building adjacent to the electrical equipment. It may be preferable to use a trailer-mounted, portable generator to provide backup power for the Booster Pump Station rather than locating a permanent generator at this site.

A more detailed evaluation of backup power requirements and specific loads that would be deemed critical in the event of loss of the main breaker into the pump station is strongly recommended during the Enhanced Engineering Analysis to optimize the sizing of these generators and associated facilities.

6.6. PIPE MATERIAL

Several materials would be suitable for this pipeline. The most common pipe types for this function and size are welded steel, ductile iron, pretensioned concrete cylinder, and high density polyethylene pipe. Final project specifications would be written for one or more of these four pipe types.

Should the pipe be steel, it would be coated and lined. The lining is usually cement mortar, although epoxy linings are occasionally used. The coating can be cement mortar, epoxy, or polyethylene tape. Cathodic protection may be used to protect the pipe from corrosion, depending on the corrosiveness of local soils. This would be determined during predesign investigations.

Should the pipe be ductile iron, it would have a cement mortar lining. The pipe would have no coating bonded to the pipe, but polyethylene sleeves would slide over the pipe for corrosion protection. Cathodic protection may be used as with steel pipe.

No additional lining or coating is used with pretensioned concrete cylinder pipe. Cathodic protection may be used as with steel pipe.

No coating or lining is necessary for high-density polyethylene pipe. The pipe itself requires no cathodic protection, but the valves and some of the other appurtenances would include ferrous metals and may require cathodic protection.

6.7. PIPELINE APPURTENANCES

The piping system would include valves at strategic locations. A valve would be provided on the two downstream sides of each "tee" in the pipeline to isolate reaches of the pipe for maintenance. Isolation valves would be installed approximately every 1,000 to 2,500 feet along the pipe where there are no "tees." The system would also include an air release valve at each high point and a blowoff at each low point. The air release valve assembly would be housed in a small aboveground enclosure located near the roadway right-of-way line. The blowoff assembly would be entirely below ground. The system would also include access ports into the pipeline at intervals of approximately 1,000 to 2,500 feet.

6.8. CONSTRUCTION CHARACTERISTICS

The pipe trench would be typically 12 feet wide for the Sacramento pipe and 7 to10 feet wide for the other partners' pipe. The trench would be 10 to 15 feet deep. Shoring would be used to maintain a narrow vertical side-wall trench and to protect workers. See **Figure 4-2** in **Chapter 4** for a typical trench cross section. A work area at least 5 feet wide on one side of the trench and at least 15 feet wide on the other side of the trench would be needed for construction. Where it is available, a larger work area of up to 40 feet on one side of the trench would be provided to facilitate construction and reduce cost. Some of the work area can be achieved through temporary lane closures during work hours.

Groundwater is high year-round along the alignment west of the Natomas East Main Drainage Canal. Extensive dewatering would be needed during construction in those areas with high groundwater, from before the trench is opened until after the trench is backfilled. Water removed from the construction area would be treated to remove sediment and discharged to the closest drainage way. A discharge permit would be needed. The dewatering method most likely to be used is a network of well points along the pipeline alignment. The wells would be drilled to several feet below the trench invert, which would be 10 to 12 feet below grade. Well spacing could vary widely. Commonly, wells would be about 100 feet apart.

Pipe bedding would be crushed rock or sand. Pipe zone backfill would be sand or crushed rock or controlled density fill (very low strength concrete). Trench zone backfill would be native material. Any native materials unsuitable for trench backfill would be hauled away to a disposal site selected by the project sponsors.

Crews would be able to install pipe of this size and depth at production rates of 100 feet of trench per day during dry weather if no problems occur. However, to account for possible delays, average production rates would probably be about 40 feet of trench per day. **Table 6-6** presents the estimated pipe lengths and construction durations for segments of the treated water pipeline. For long reaches, it is assumed that the contractor would use multiple headings, thus reducing the construction duration. A contract period 40 to 60 workdays longer than the construction period would be needed to allow for mobilization, demobilization, punchlist work, and weather delays. Typical workdays would be from 7:00 a.m. to 3:30 p.m. Monday through Friday, with occasional work as late as 7:00 p.m. and occasional work on Saturday.

Table 6-6 Estimated Construction Duration for the Treated Water Pipelines

	Pipe Length (feet)	Trench Length (feet)	Construction Duration (work days)	Contract Period (work days)	Contract Period (calendar days)
From a WTP site near the middle of potential sites (about 2.6 miles from the Intake) to Sacramento turnout	36,000	36,000	450 (2 headings)	500	800
From a WTP site near the middle of potential sites (about 2.6 miles from the Intake) to PCWA Placer Vineyards turnout	50,000	50,000	625 (2 headings)	675	1,100
Along Baseline Road from PCWA Placer Vineyards turnout to tee at Old Walerga Road	19,000	19,000	475	525	840
Along Walerga Road from Baseline Road to Antelope Road	18,000	18,000	450	500	800
Along Baseline Road from Old Walerga Road to Fiddyment Road	1,600	1,600	40	80	130
Along Fiddyment Road from Baseline Road to Athens Road	33,000	33,000	400 (2 headings)	450	720

The construction operation could use a number of different combinations of equipment. One possible setup would include one or two excavators to excavate the trench, place pipe bedding and pipe zone backfill and set the pipe; a front end loader to move soil around the work site and load trucks; a dozer or tractor to move trench backfill into place; a large compactor and smaller walk-behind compactors; to six end dump trucks to haul soil to and from the work site; and miscellaneous trucks to deliver materials and imported fill. Crew size would be 6 to 10 people, not including truck drivers. The crew superintendent and the contractor's project manager and field engineer may be local staff or, if the contractor is not a local contractor, may be brought in from outside the local area.

The number of truck trips to and from the construction site each day would vary depending on how much of the native soil can be used for backfill. If all the backfill can be native material taken from the trench and stored at the work area, only about 11 truck trips would be made to haul away excess material and 11 more truck trips to haul in imported material on an average day. Should the native material be unsuitable for backfill or inadequate space at the work site to store the material until the trench is ready for backfill, the number of truck trips would go up to about 23 truck trips to bring in material and about 23 truck trips to haul away material.

Trucks hauling materials to and from the construction site would have loads that keep their weight below highway load limits. Trucks hauling soil, rock, or sand to and from the job site would haul from 5 to 10 cubic yards of material in each load. Loads for other trucks would vary depending on what is being hauled, but would always be below H-20 load limits.

Safety on the construction site would be the responsibility of the construction contractor. The construction contractor would have a company safety program and a job-specific safety program, administered by a project safety officer. Typical procedures would include weekly safety meetings with the construction crew and hazard analyses prepared before the beginning of each new operation. A traffic control plan would be prepared by the construction contractor and reviewed by Sacramento County to make sure traffic is safely routed around the work site. OSHA and Cal-OSHA standards would apply for all work.

For most operations, no particularly noisy equipment is anticipated for the construction work (e.g., no pile driving). The contractor may elect to drive soldier piles and/or sheet piles for shoring of the trench or the jacking and receiving pits. These pile driving operations would be short term. Typical noise would include noise from trucks and diesel-powered equipment. The work would comply with all county noise ordinances.

The construction contractor would have a staging area for field offices and to temporarily park equipment and supplies. This area would be 1 to 5 acres in size. A site has not been selected for this staging area. A 2- to 10-acre site would be used for disposal of excess material removed from the trench. Some material would be stockpiled only temporarily at the disposal site and then used later for backfill. Other material would be permanently placed at the disposal site. A grading permit would be obtained for the disposal site. Work at the disposal site would comply with all county requirements, including the grading ordinance and sedimentation and erosion control requirements. A location has not yet been selected for this disposal site.

The treated water pipeline crosses several drainage ways. A stream alteration permit would be requested from CDFG for each crossing. The permit may not allow using open-cut trenching to install the pipe across the stream; instead tunneling may be required. A pressure balance tunneling technology would be used because the tunnel would be below groundwater levels. The tunneling would involve an approximately 15 feet wide by 30 feet long by 25 feet deep jacking pit on one side of the stream and a smaller receiving pit on the other side.

The construction contract documents would include a general SWPPP. The construction contractor would be required to submit a specific, more detailed SWPPP. The general plan would outline minimum requirements that must be met to minimize erosion and control sediments. The general and specific SWPPP would comply with the county sediment and erosion control ordinances. Typical best management practices that would be used include the following:

- Covering all exposed slopes and stockpiles with plastic, straw, or hydroseed
- Placing silt fences at the downstream side of all work areas
- Placing a sediment filter in each drop inlet
- Sweeping all work areas frequently
- Constructing sediment ponds in key locations
- Placing waddles or hay bales across steep, disrupted slopes
- Constructing gravel driveways at each work site exit
- Placing waddles or straw bales around the open trench work area

6.9. OPERATING CHARACTERISTICS

Pipelines require very little operations or maintenance. Cathodic protection systems, if used, must be checked once a year. Valves should be exercised every few years. When the pipeline gets older (e.g., 50 years old or more), occasional pipeline breaks may occur depending on how corrosive the soils are and how well the cathodic protection system is maintained. These breaks would necessitate pipe repairs.

Turnouts would include flow monitoring and a flow control valve. Regular operation, monitoring, and repair of the turnouts would be needed. Five to ten people would be involved in pipeline maintenance but none would work full time on this pipeline.

Regular maintenance of the booster pump station would be needed. This would include exercising the backup generator, monitoring the status of equipment at the pump station, and repairing any damaged parts. It is estimated that maintenance and repair and operation of this pump station would take the equivalent of about 20 percent of one person's time.

No hazardous materials would be used for operation of the pipelines and Booster Pump Station. No regular large truck traffic would be associated with operations and maintenance of the pipelines and Booster Pump Station.

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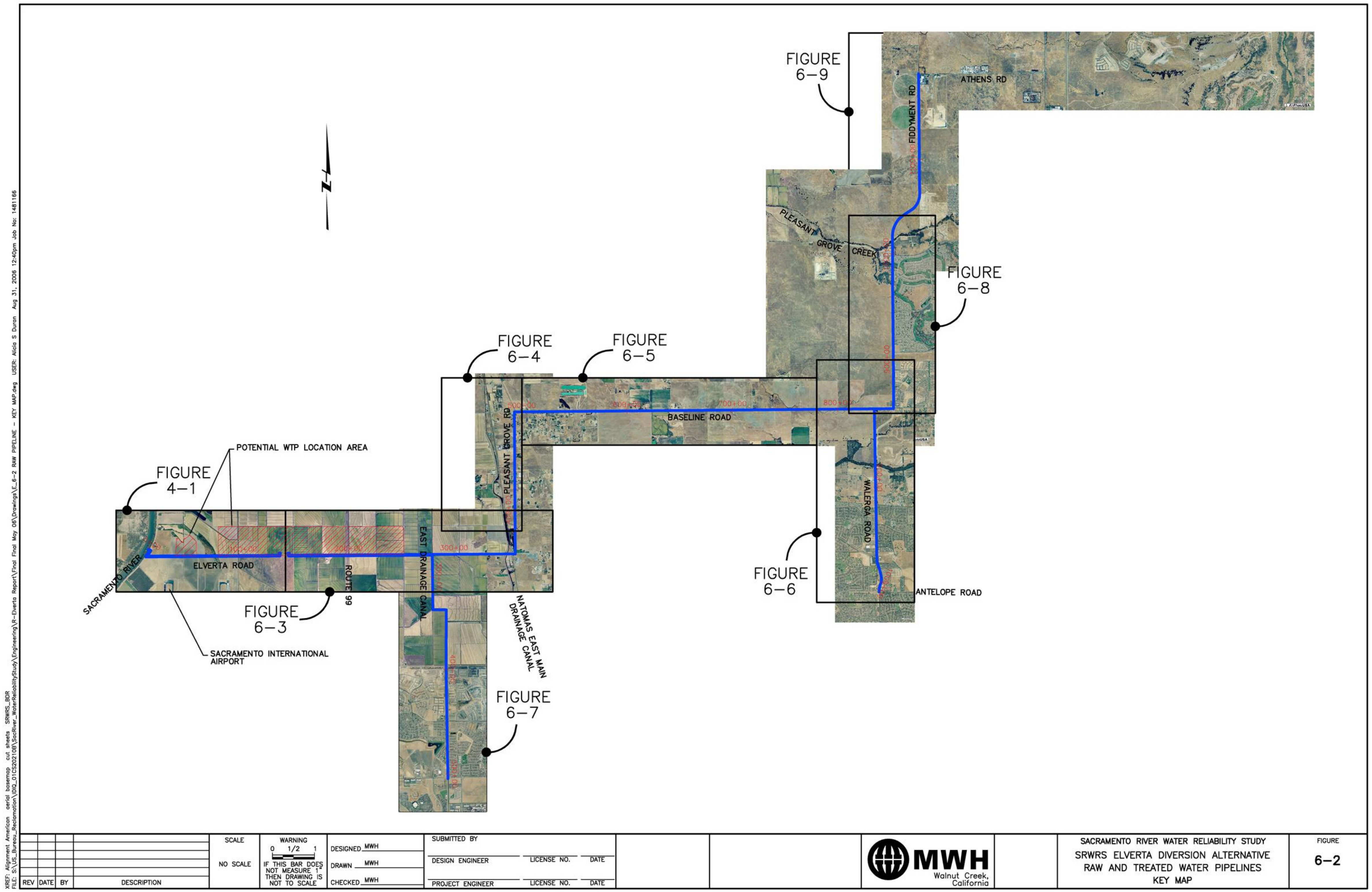

Figure 6-2 SRWRS Elverta Diversion Alternative Sacramento Pipeline Key Map

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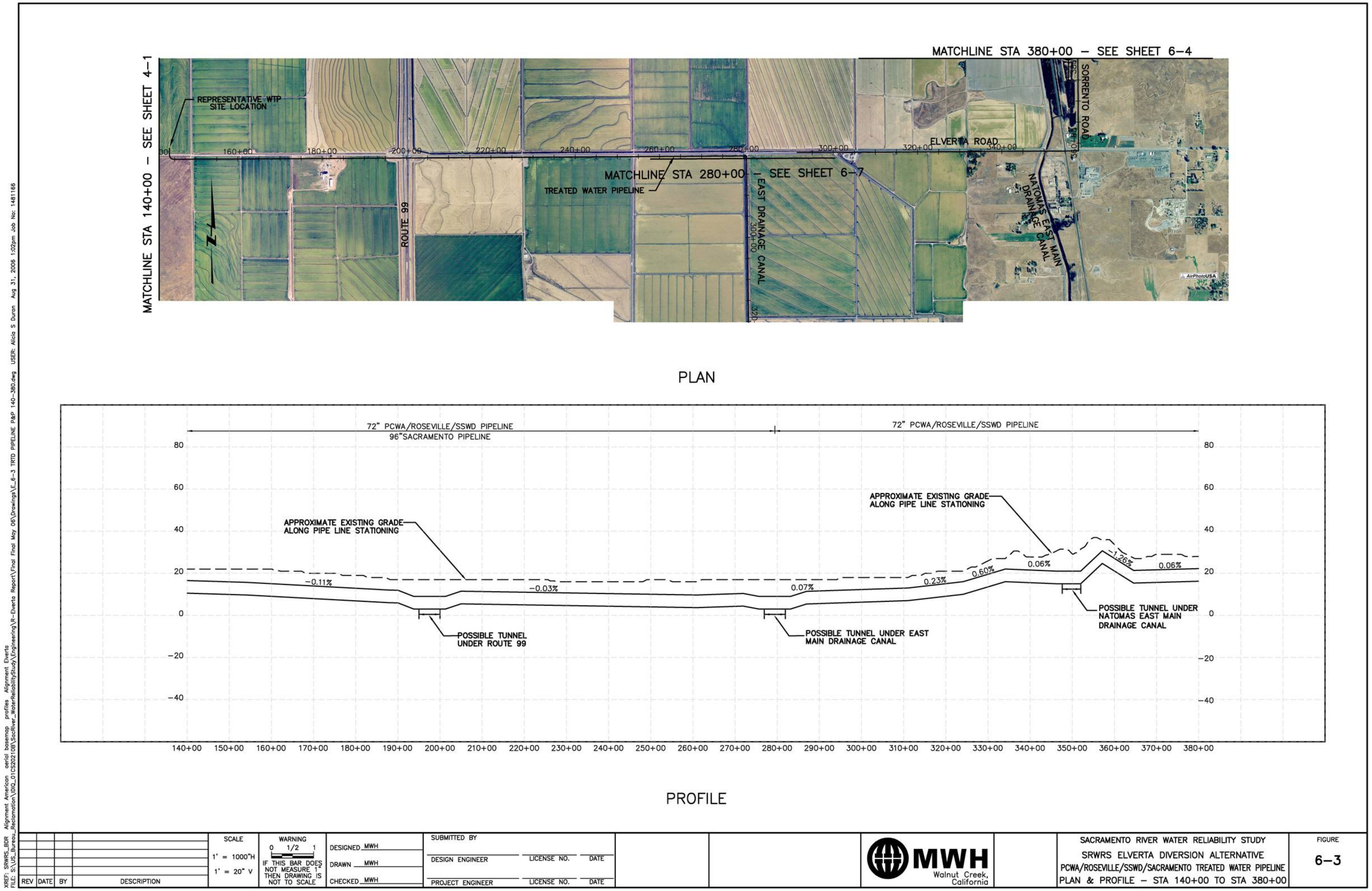

Figure 6-3 SRWRS Elverta Diversion Alternative Treated Water Pipeline Plan and Profile – STA 140+00 to STA 380+00

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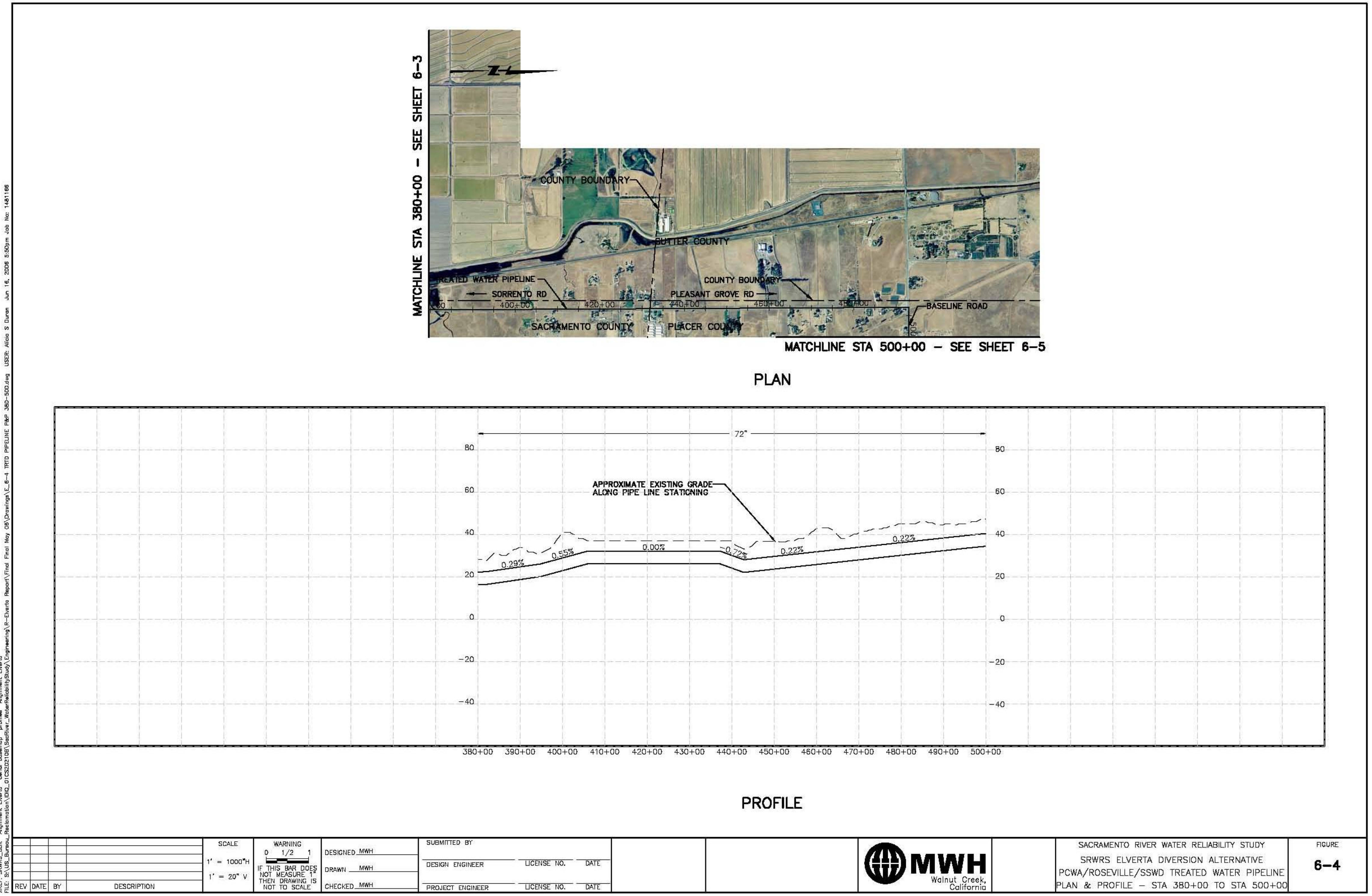

Figure 6-4 SRWRS Elverta Diversion Alternative Treated Water Pipeline Plan and Profile – STA 380+00 to STA 500+00

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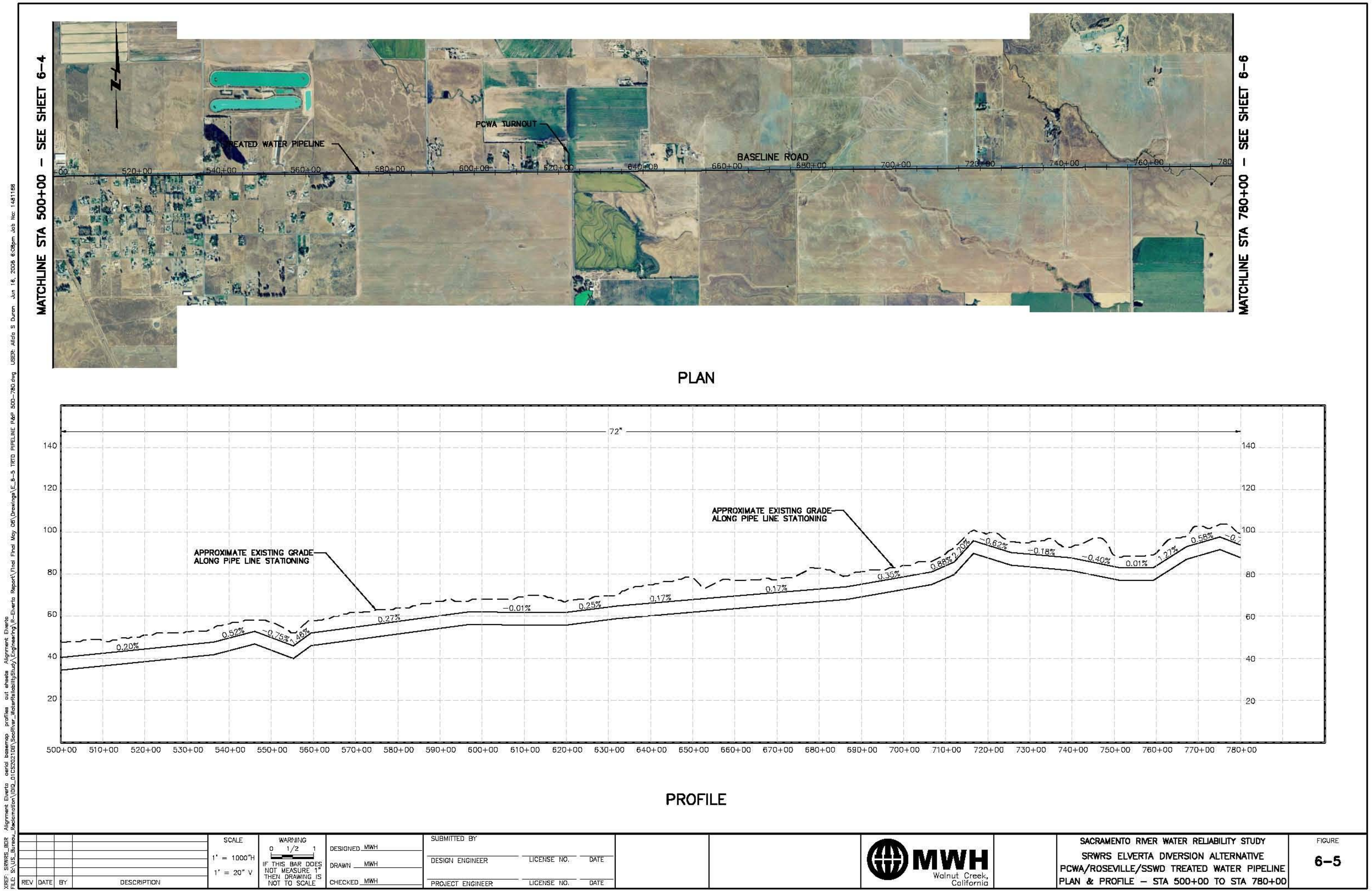

Figure 6-5 SRWRS Elverta Diversion Alternative Treated Water Pipeline Plan and Profile – STA 500+00 to STA 780+00

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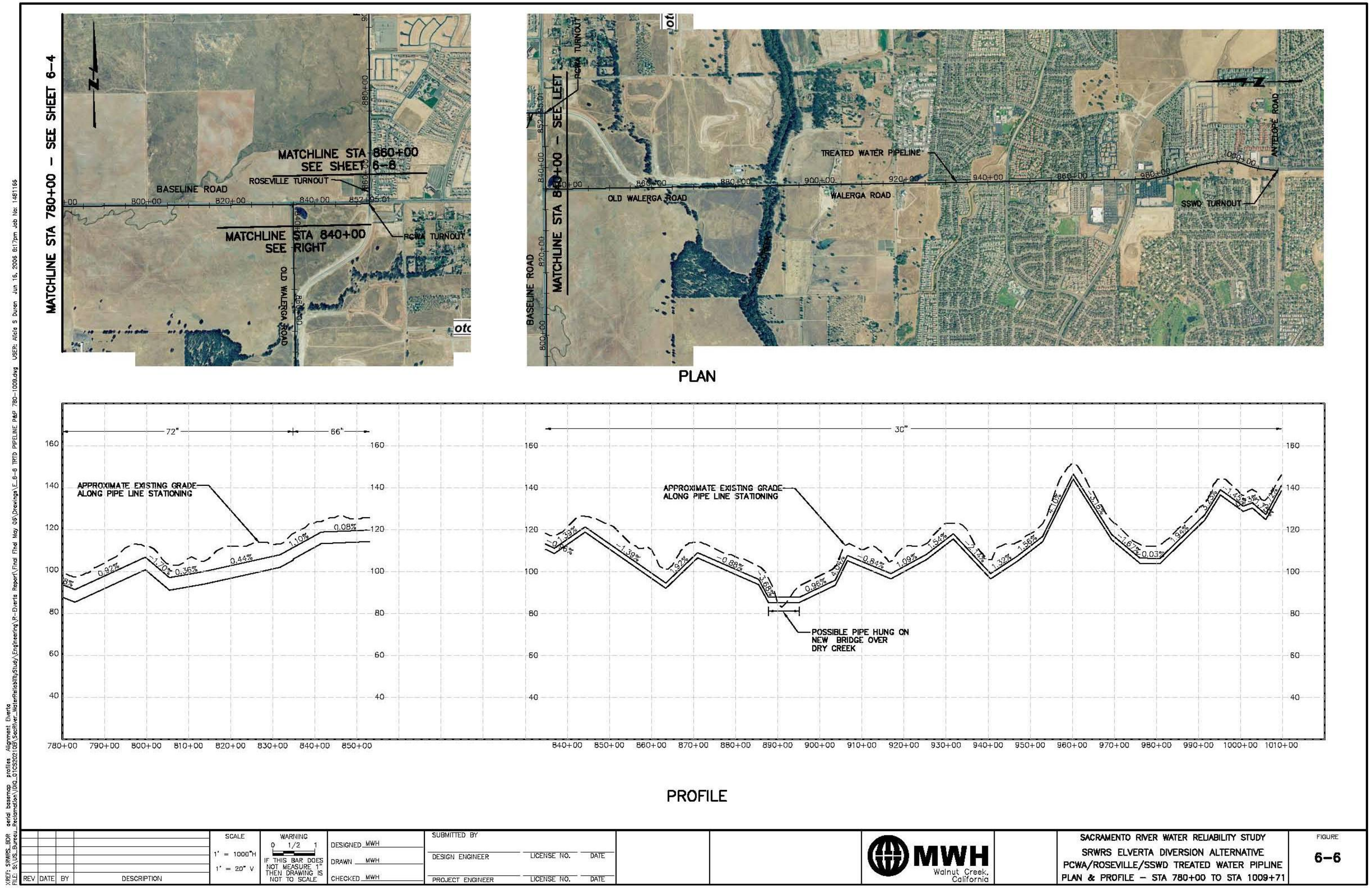

Figure 6-6 SRWRS Elverta Diversion Alternative PCWA/Roseville/SSWD Treated Water Pipeline Plan and Profile – STA 780+00 to STA 1009+71

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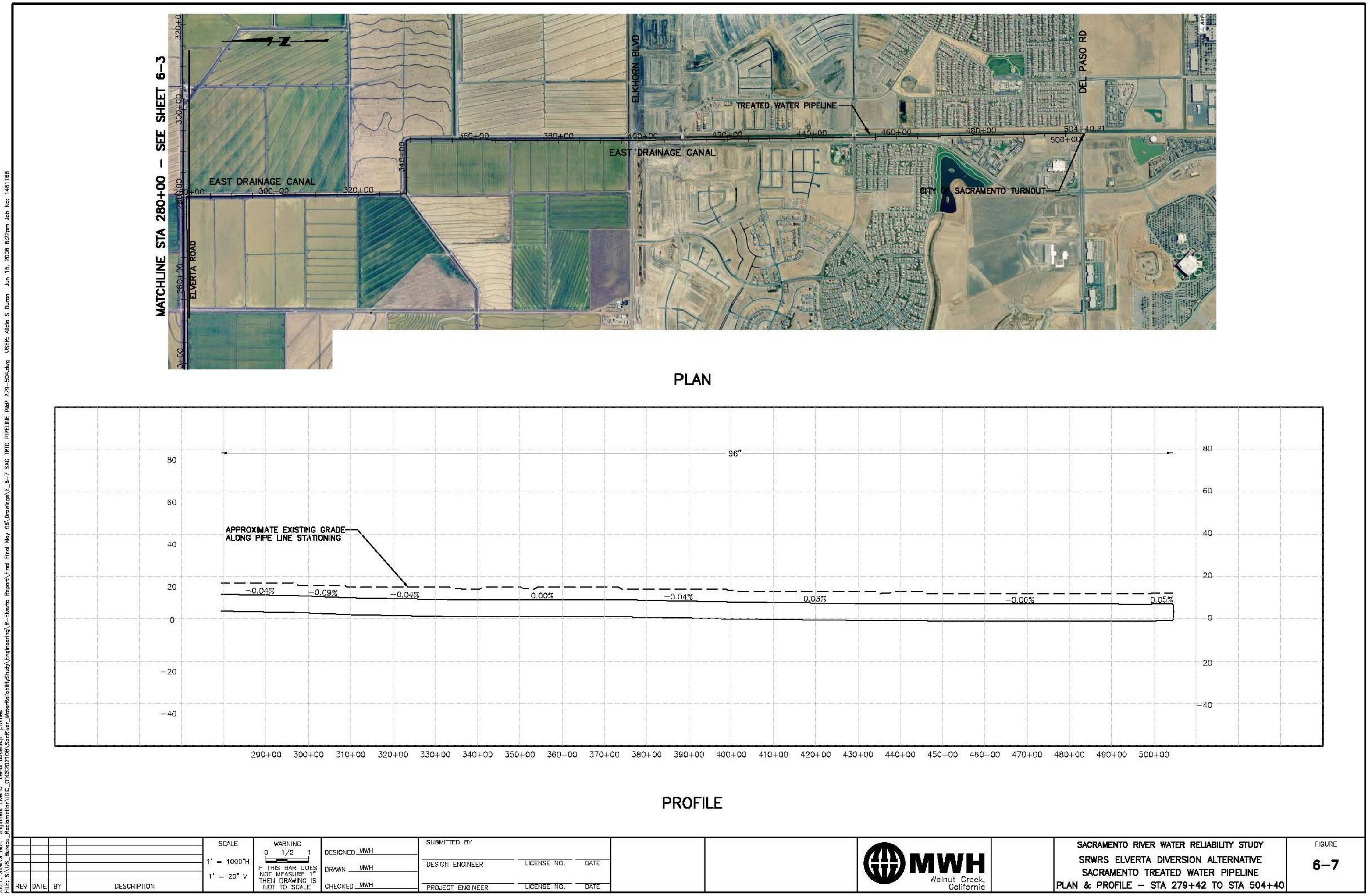

Figure 6-7 SRWRS Elverta Diversion Alternative Sacramento Treated Water Pipeline Plan and Profile – STA 279 to STA 504+40

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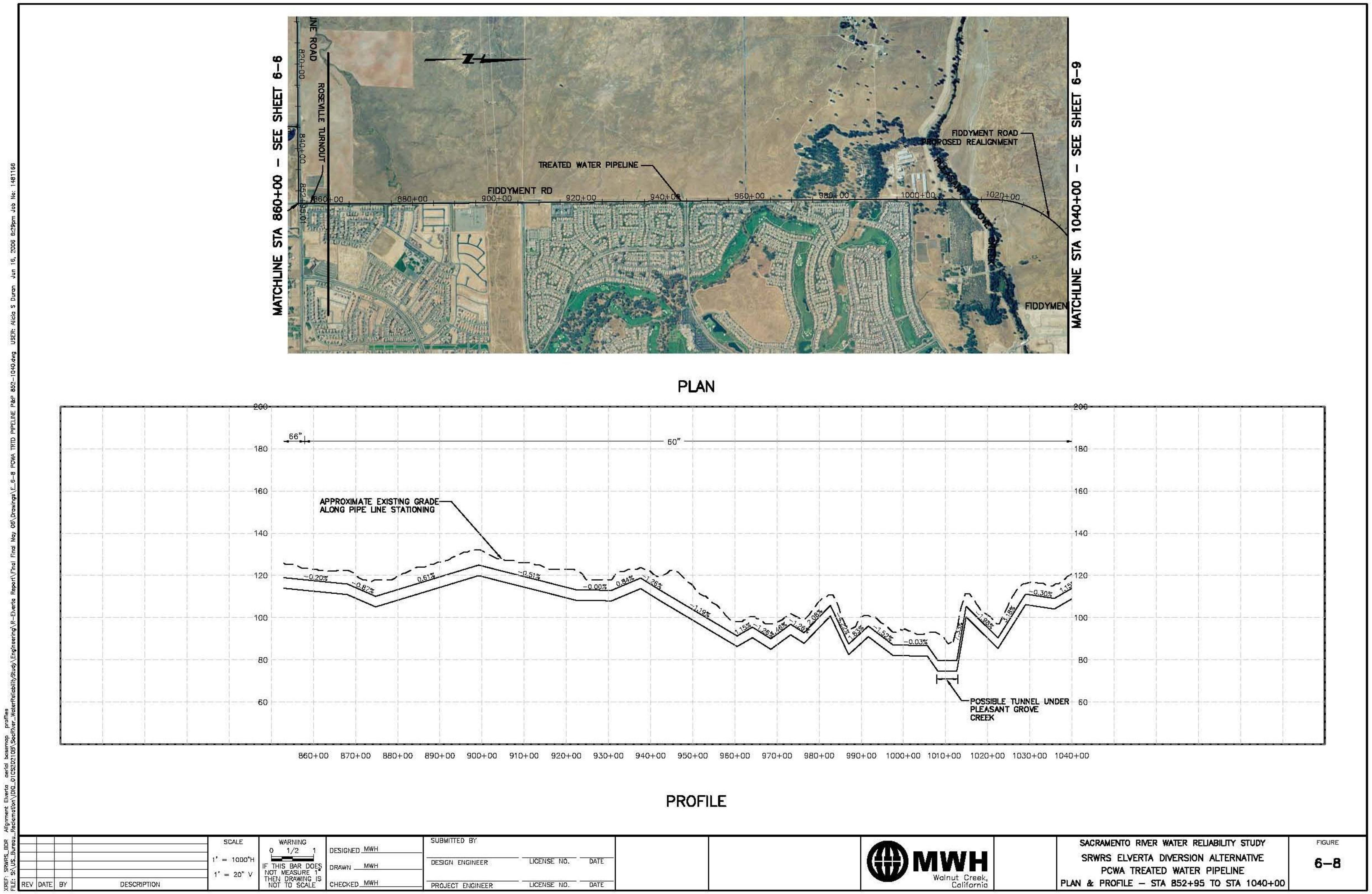

Figure 6-8 SRWRS Elverta Diversion Alternative PCWA Treated Water Pipeline Plan and Profile – STA 852+95 to STA 1040+00

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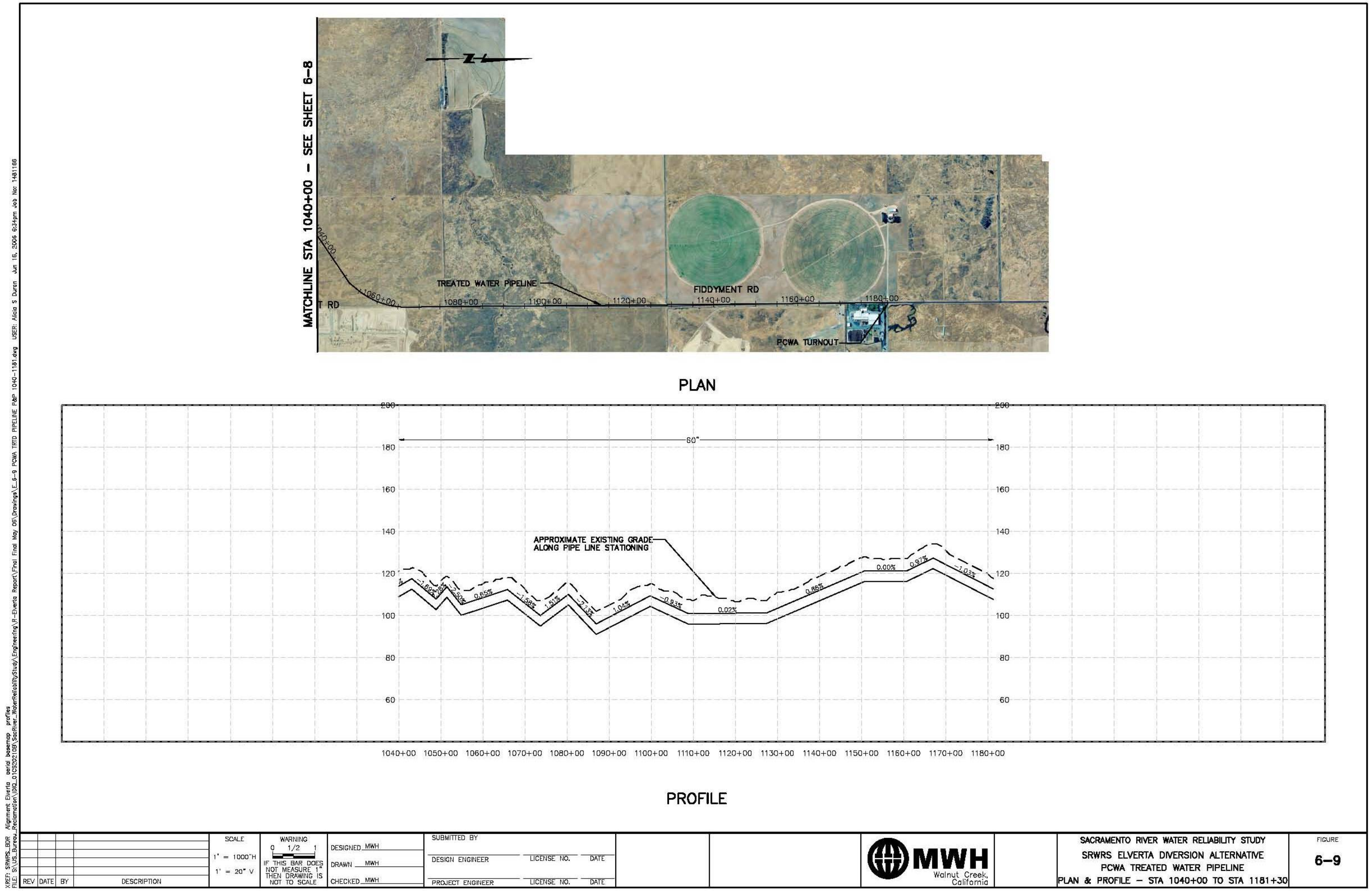

Figure 6-9 SRWRS Elverta Diversion Alternative PCWA Treated Water Pipeline Plan and Profile – STA 1040+00 to STA 1181+30

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CHAPTER 7 COST ESTIMATE

A feasibility-level cost estimate has been developed for the SRWRS Elverta Diversion Alternative. The cost estimate has been separated into four main sections: Intake Improvements, Raw Water Conveyance, Water Treatment, and Treated Water Conveyance. An additional section for the Joint SRWRS-ABFSHIP Elverta Diversion subalternative, which includes NMWC participation, has been provided. To simplify evaluation and comparison, quantities used in the cost estimate assume that the WTP would be constructed on Elverta Road at a site located near the middle of the potential sites (approximately 2.6 miles from the intake). Final selection of the preferred WTP site will not occur until the next phase of the project; however, cost differences between the three potential sites are not expected to vary significantly given this report's scope and level of detail. Costs for easements and land purchases, as well as future advanced oxidation processes, have not been included in the estimate.

Costs have been developed for July 2006 cost basis and then escalated to the estimated midpoint of construction in 2012. Costs for the proposed intake have been determined by starting with actual costs for the recently constructed Sacramento River WTP intake and modifying those costs to reflect an increased flow rate, a slightly longer bridge, and a reduced architectural effort. Raw and treated water pipeline costs have been developed using current pipeline construction pricing and incorporating additional project-specific costs for such elements as tunnel crossings, high groundwater conditions, and rock conditions. Water treatment costs are based on current and historical WTP construction costs using a cost-per-mgd basis and including additional site-specific costs such as foundation piles.

Total costs for engineering, environmental, administration, and legal services have been estimated at 30 percent of construction costs. In addition, a 20 percent estimating contingency has been included.

Cost estimate detail is included in **Table 7-1**. The total escalated project cost, excluding NMWC, has been estimated at $1.123 billion (SRWRS Elverta Diversion Alternative). Including NMWC adds $43 million, for a total project cost of $1.166 billion (Joint SRWRS-ABFSHIP Elverta Diversion Alternative). A distribution of costs per cost-sharing partner, based only on percentage of flow capacity, has also been included for reference.

Table 7-1 SRWRS Elverta Diversion Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
Base Scope								
1		**INTAKE IMPROVEMENTS**						
	A	***Sacramento River Intake - RM 74.6***						
	1)	Sacramento River Intake Structure - CIP	MGD	235	$ 145,000	$ 34,075		Factored from Sac River WTP SOV - Balfour Beatty
	2)	Mobe/Demobe & Contr General Conds	MGD	235	$ 18,000	$ 4,230		Factored from Sac River WTP SOV - Balfour Beatty
	3)	Allowances	LS	1	$ -	$ -		Included
	4)	Temporary Controls	LS	1	$ -	$ -		Included
	5)	Demonstration & Training	LS	1	$ -	$ -		Included
						Item Total:	**$ 38,300**	Cost Basis 7/06
2		**RAW WATER CONVEYANCE**						
	A	***Sacramento River Intake - To N. Natomas WTP***						
	1)	C200 - Dual 78" Pipelines at Levee Rd	LF	1,500	$ 1,700	$ 2,550		Next to Levee, Placed In Fill, 4' Cover
	2)	Garden Hwy/Levee Improvements	LS	1	$ 606,000	$ 606		Raise Levee, Construct Temp Detour, Repave Grd Hwy, 1000'
	3)	Install Pipe on Raised Levee at Grd Hwy Xing	LF	600	$ 1,900	$ 1,140		Install Dbl Pipes at Raised Grade
	4)	Jacob's Slough Microtunnel	LF	500	$ 1,900	$ 950		Stream Crossing
	5)	C200 - Dual 78" Pipelines at Elverta Rd.	LF	13,400	$ 1,700	$ 22,780		North Side of Elverta Rd, in ROW, High GW
	6)	Cathodic Protection	LF	13,400	$ 20	$ 268		Allowance
	7)	Isolation Valves	EA	27	$ 60,600	$ 1,636		@ 1,000'
	8)	Air Release / Blowoff Valves	EA	6	$ 12,100	$ 73		@ High/Low Points
	9)	Access Manways	EA	27	$ 12,100	$ 327		@ 1,000'
	10)	Increased Dewatering Allowance	LF	13,400	$ 60	$ 804		Well Point System, See LNWI Program
	11)	Increased Shoring Allowance - Sheet Piles	LF	13,400	$ 610	$ 8,174		Drive/Pull Sheet Piles 25' Driving Depth, 75% Reuse
	12)	Increased Traffic Controls - Elverta Rd.	LF	13,400	$ 20	$ 268		One Lane w/ Flagmen Control
	13)	Pavement Restoration - Elverta Rd.	LF	6,700	$ 60	$ 402		Assume 50% of Alignment on Elverta Rd
	14)	Mobe/Demobe & Contr General Conds	LS	1	$ 3,200,000	$ 3,200		Included at 8% of Direct Costs
	15)	Allowances	LS	1	$ 1,212,000	$ 1,212		Unknown Geology/Utilities
	16)	Temporary Controls	LS	1	$ 606,000	$ 606		Unknown Flood Controls/Envir Protections
	17)	Demonstration/Training/Cleanup	LS	1	$ 180,000	$ 100		Includes Startup & Testing Program
						Item Total:	**$ 45,100**	Cost Basis 7/06

Table 7-1 SRWRS Elverta Diversion Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
3		**WATER TREATMENT**						
	A	***N. Natomas Water Treatment Plant***	235	MGD				Conventional Filtration w/ Chlorine Disinfection
	1)	Plant Construction - CIP	MGD	235	$ 910,000	$ 213,850		Historical Pricing
	2)	Booster Pump Stations - 2 ea	HP	16,350	$ 1,500	$ 24,525		Historical Pricing
	3)	Increase for Foundation System	LS	1	$ 12,120,000	$ 12,120		Driven Conc Piles at 10' Spacing 60' Deep
	4)	Mobe/Demobe & Contr General Conds	LS	1	$ -	$ -		Included
	5)	Allowances	LS	1	$ 1,210,000	$ 1,210		Dewatering/Environmental
	6)	Temporary Controls	LS	1	$ 1,210,000	$ 1,210		Environmental
	7)	Demonstration/Training/Cleanup	LS	1	$ -	$ -		Included
							$ 252,900	Cost Basis 7/06
4		**TREATED WATER CONVEYANCE**						
	A	***City of Sacramento Pipeline***						
	1)	C200 96" from WTP to existing system	LF	36,000	$ 1,000	$ 36,000		8 Miles, Elverta Rd to existing system
	2)	Microtunnel - Major (1 ea)	LF	800	$ 1,500	$ 1,200		EDC
	3)	Microtunnel - Average (2 ea)	LF	1,000	$ 1,000	$ 1,000		Surface Streets at 500'
	4)	Cathodic Protection	LF	36,000	$ 20	$ 720		Allowance
	5)	Isolation Valves	EA	42	$ 60,600	$ 2,545		@ 1,000'
	6)	Air Release / Blowoff Valves	EA	8	$ 12,100	$ 97		@ High/Low Points
	7)	Access Manways	EA	42	$ 12,100	$ 508		@ 1,000'
	8)	Increased Dewatering Allowance	LF	36,000	$ 50	$ 1,800		Well Point System, See LNWI Program
	9)	Increased Shoring Allowance - Sheet Piles	LF	36,000	$ 600	$ 21,600		Drive/Pull Sheet Piles 25' Driving Depth, 75% Reuse
	10)	Increased Traffic Controls - Elverta Rd.	LF	20,000	$ 20	$ 400		One Lane w/ Flagmen Control
	11)	Pavement Restoration - Elverta Rd.	LF	10,000	$ 100	$ 1,000		Assume 50% of Alignment on Elverta Rd.
	12)	Mobe/Demobe & Contr General Conds	LS	1	$ 4,680,000	$ 4,680		Included at 8% of Direct Costs
	13)	Allowances	LS	1	$ 1,210,000	$ 1,210		Unknown Geology/Utilities
	14)	Temporary Controls	LS	1	$ 610,000	$ 610		Unknown Flood Controls/Envir Protections
	15)	Demonstration/Training/Cleanup	LS	1	$ 180,000	$ 100		Includes Startup & Testing Program
	16)	Turnout/Distribution Facilities	LS	1	$ 300,000	$ 300		End Point Allowance
						Item Total:	**$ 73,800**	Cost Basis 7/06

Table 7-1 SRWRS Elverta Diversion Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
	B	***PCWA, SSWD, Roseville Pipeline - Reach 1***						
	1)	C200 72" from WTP to Baseline Rd/Old Walerga Rd	LF	69,000	$ 800	$ 55,200		8.3 Miles, Elverta Rd to Baseline Rd
	2)	Microtunnel - Major (4 ea)	LF	3,200	$ 1,500	$ 4,800		Major Street/Hwy/RR Xings at 800'
	3)	Microtunnel - Average (4 ea)	LF	2,400	$ 1,000	$ 2,400		Minor Street/Canal/RR Xings at 600'
	4)	Cathodic Protection	LF	69,000	$ 20	$ 1,380		Allowance
	5)	Isolation Valves	EA	69	$ 60,600	$ 4,181		@ 1,000'
	6)	Air Release / Blowoff Valves	EA	15	$ 12,100	$ 182		@ High/Low Points
	7)	Access Manways	EA	69	$ 12,100	$ 835		@ 1,000'
	8)	Increased Dewatering Allowance	LF	69,000	$ 40	$ 2,760		Well Point System, See LNWI Program
	9)	Increased Shoring Allowance - Sheet Piles	LF	34,500	$ 600	$ 20,700		Drive/Pull Sheet Piles 25' Driving Depth, 75% Reuse, 50% of length
	10)	Increased Traffic Controls	LF	69,000	$ 20	$ 1,380		One Lane w/ Flagmen Control
	11)	Pavement Restoration - Elverta/Baseline	LF	69,000	$ 40	$ 2,760		Assume 50% of alignment on Elverta Rd
	12)	Mobe/Demobe & Contr General Conds	LS	1	$ 6,760,000	$ 6,760		Included at 8% of Direct Costs
	13)	Allowances	LS	1	$ 1,820,000	$ 1,820		Unknown Geology/Utilities/Environmental Allowance
	14)	Temporary Controls	LS	1	$ 610,000	$ 610		Unknown Flood Controls/Envir Protections
	15)	Demonstration/Training/Cleanup	LS	1	$ 182,000	$ 100		Includes Startup & Testing Program
	16)	Turnout/Distribution Facilities	LS	2	$ 303,000	$ 350		End Point Allowance
						Item Total:	**$ 106,200**	Cost Basis 7/06
	C	***PCWA, Roseville Pipeline - Reach 2***						
	1)	C200 66" from Old Walerga Rd to Fiddyment	LF	1,600	$ 700	$ 1,120		2.6 Miles,
	2)	Microtunnel - Major (1 ea)	LF	500	$ 1,200	$ 600		Major Street/Hwy/RR Xings at 800'
	3)	Microtunnel - Average (2 ea)	LF	1,000	$ 700	$ 700		Minor Street/Canal/RR Xings at 600'
	4)	Cathodic Protection	LF	1,600	$ 20	$ 32		Allowance
	5)	Isolation Valves	EA	2	$ 60,600	$ 121		@ 1,000'
	6)	Air Release / Blowoff Valves	EA	5	$ 12,100	$ 61		@ High/Low Points
	7)	Access Manways	EA	2	$ 12,100	$ 24		@ 1,000'
	8)	Increased Rock Allowance	LF	640	$ 50	$ 32		Well Point System, See LNWI Program
	9)	Increased Shoring Allowance - Sheet Piles	LF	0	$ 610	$ -		Drive/Pull Sheet Piles 25' Driving Depth, 75% Reuse
	10)	Increased Traffic Controls	LF	1,600	$ 120	$ 192		One Lane w/ Flagmen Control
	11)	Pavement Restoration	LF	1,600	$ 60	$ 96		Assume 50% of alignment on Elverta Rd
	12)	Mobe/Demobe & Contr General Conds	LS	1	$ 210,000	$ 210		Included at 8% of Direct Costs
	13)	Allowances	LS	1	$ 300,000	$ 300		Unknown Geology/Utilities
	14)	Temporary Controls	LS	1	$ 242,400	$ 242		Unknown Flood Controls/Envir Protections
	15)	Demonstration/Training/Cleanup	LS	1	$ 60,600	$ 61		Includes Startup & Testing Program
	16)	Turnout/Distribution Facilities	LS	1	$ 121,000	$ 121		End Point Allowance
						Item Total:	**$ 3,900**	Cost Basis 7/06

Table 7-1 SRWRS Elverta Diversion Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
	D	***PCWA Pipeline - Reach 3***						
	1)	C200 60" from Baseline to Athens	LF	33,000	$ 520	$ 17,160		6 Miles,
	2)	Microtunnel - Major (2 ea)	LF	1,200	$ 970	$ 1,164		Major Street/Hwy/RR Xings at 800'
	3)	Microtunnel - Average (2 ea)	LF	1,000	$ 730	$ 730		Minor Street/Canal/RR Xings at 600'
	4)	Cathodic Protection	LF	33,000	$ 20	$ 660		Allowance
	5)	Isolation Valves	EA	33	$ 60,610	$ 2,000		@ 1,000'
	6)	Air Release / Blowoff Valves	EA	5	$ 12,120	$ 61		@ High/Low Points
	7)	Access Manways	EA	33	$ 12,120	$ 400		@ 1,000'
	8)	Increased Rock Conditions Allowance	LF	16,500	$ 40	$ 660		Rip/Blast Trench Profile
	9)	Increased Shoring Allowance - Sheet Piles	LF	0	$ 610	$ -		Drive/Pull Sheet Piles 25' Driving Depth, 75% Reuse
	10)	Increased Traffic Controls	LF	21,978	$ 20	$ 440		One Lane w/ Flagmen Control
	11)	Pavement Restoration	LF	21,978	$ 60	$ 1,319		Assume 50% of alignment on Elverta Rd.
	12)	Mobe/Demobe & Contr General Conds	LS	1	$ 1,722,000	$ 1,722		Included at 8% of Direct Costs
	13)	Allowances	LS	1	$ 606,000	$ 606		Unknown Geology/Utilities
	14)	Temporary Controls	LS	1	$ 303,000	$ 303		Unknown Flood Controls/Envir Protections
	15)	Demonstration/Training/Cleanup	LS	1	$ 90,900	$ 91		Includes Startup & Testing Program
	16)	Booster Pump Station	HP	885	$ 1,800	$ 1,593		PCWA
	18)	Turnout/Distribution Facilities	LS	1	$ 303,000	$ 303		End Point Allowance
						Item Total:	**$ 29,200**	Cost Basis 7/06
	E	***SSWD Pipeline - Reach 1***						
	1)	C200 30" Along Walerga from Baseline	LF	17,424	$ 330	$ 5,750		3.3 Miles
	2)	Microtunnel - Major	LF	0	$ 970	$ -		Major Street/Hwy/RR Xings at 800'
	3)	Microtunnel - Average (1 ea)	LF	500	$ 730	$ 365		Minor Street/Canal/RR Xings at 600'
	4)	Cathodic Protection	LF	17,424	$ 20	$ 348		Allowance
	5)	Isolation Valves	EA	16	$ 60,610	$ 970		@ 1,000'
	6)	Air Release / Blowoff Valves	EA	4	$ 12,120	$ 48		@ High/Low Points
	7)	Access Manways	EA	16	$ 12,120	$ 194		@ 1,000'
	8)	Increased Rock Conditions Allowance	LF	17,424	$ 30	$ 523		Drill/Shoot Trench
	9)	Increased Shoring Allowance - Sheet Piles	LF	0	$ 610	$ -		Drive/Pull Sheet Piles 25' Driving Depth, 75% Reuse
	10)	Increased Traffic Controls	LF	11,604	$ 20	$ 232		One Lane w/ Flagmen Control
	11)	Pavement Restoration	LF	11,604	$ 60	$ 696		Assume 50% of alignment on Elverta Rd.
	12)	Mobe/Demobe & Contr General Conds	LS	1	$ 639,000	$ 639		Included at 8% of Direct Costs
	13)	Allowances	LS	1	$ 303,040	$ 303		Unknown Geology/Utilities
	14)	Temporary Controls	LS	1	$ 60,610	$ 61		Unknown Flood Controls/Envir Protections
	15)	Demonstration/Training/Cleanup	LS	1	$ 60,610	$ 61		Includes Startup & Testing Program
	16)	Turnout/Distribution Facilities	LS	1	$ 242,430	$ 242		End Point Allowance
						Item Total:	**$ 10,400**	Cost Basis 7/06
						Total :	**$ 223,500**	

Table 7-1 SRWRS Elverta Diversion Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
Base Scope Cost Summary								
	a)	Item/Task Direct Costs					**$ 559,800**	Items 1-4
	b)	Escalation to Midpoint of Construction					**$ 160,200**	n=6 or 2012 at 6.0%/Year
		Subtotal (Direct Costs):					**$ 720,000**	
	c)	Engineering/Environmental/Administration Legal Services Allowance	30%				**216,000**	Allowance
		Subtotal (Direct/Indirect Costs):					**$ 936,000**	
	d)	Program Contingency	20%				**187,000**	
		Engineers' Opinion of SRWRS-Elverta Diversion Alternative Total Capital Costs:					**$ 1,123,000**	Cost Escalated to 7/12, Midpoint of Construction

Table 7-1 SRWRS Elverta Diversion Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
Additive Alternative No. 1 -Joint SRWRS-ABFSHIP								
Elverta Diversion Alternative			MGD	371				
		Scope: Provide Additional Raw Water Intake						
		Pumping Capacity and Canal Improvements						
	1)	Additional Intake Capacity at Sac River	MGD	136	$ 121,000	$ 16,456		Use 2/3's of Item's 1 Costs on an mgd Basis
	2)	Energy Dissipation Structure	LS	1	$ 420,000	$ 420		Includes Valves/Slide Gates
	3)	Relocate Canal	MI	2	$ 680,000	$ 1,088		From 2000 Report
	4)	Mobe/Demobe & Contr General Conds	LS	1	$ 1,257,000	$ 1,257		Factored at 5% of Totals
	5)	Allowances	LS	1	$ -	$ -		Included
	6)	Temporary Controls	LS	1	$ -	$ -		Included
	7)	Demonstration & Training	LS	1	$ 60,610	$ 61		Allowance
							$ 19,300	Cost Basis 7/06
Alternative Summary								
	a)	Item/Task Direct Costs					$ 19,300	
	b)	Escalation to Midpoint of Construction					$ 8,100	n=6 or 2012 at 6.0%/Year
		Subtotal (Direct Costs):					$ 27,400	
	c)	Engineering/Environmental/Administration	30%				8,200	
		Legal Services Allowance						
		Subtotal (Direct/Indirect Costs):					$ 35,600	
	d)	Program Contingency	20%				7,100	
		Engineers' Opinion of Joint SRWRS-ABFSHIP Elverta Diviersion Alternative Additive Capital Costs:					**$ 43,000**	Cost Escalated to 7/12, Midpoint of Construction

Table 7-1 SRWRS Elverta Diversion Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
Partner Share Summary								
	a)	City of Sacramento					**$ 551,200**	
	b)	Placer County Water Agency (PCWA)					**$ 396,500**	
	c)	Sacramento Suburban Water District (SSWD)					**$ 114,500**	
	d)	City of Roseville (Roseville)					**$ 61,000**	
							$ 1,123,000	Cost Escalated to 7/12, Midpoint of Construction
	e)	Natomas Mutual Water Company (NMWC)					**$ 43,000**	
		Engineers' Opinion of Joint SRWRS-ABFSHIP Elverta Diviersion Alternative Total Capital Costs:					**$ 1,166,000**	Cost Escalated to 7/12, Midpoint of Construction

Notes/Comments

- Cost basis = 7/06
- Costs for easements, land purchase, or future advanced oxidation processes have been excluded.
- Includes City of Sacramento Fluoridation Dosing Station
- Pricing predicated on competitive market conditions (+4 bidders/trade).
- Pricing excludes any labor premium for overtime conditions.
- Pricing excludes a change order contingency.
- Pricing excludes an allowance for design/CM oversight or field inspections services.
- Pricing excludes any financing or cost-of-money expenses.
- Pricing excludes costs for obtaining required bldg permits or fees.

Key:

CIP – cast-in-place concrete	LNWI – Lower Northwest Interceptor	ROW – right-of-way
CM – Construction Management	ls – lump sum	RR – railraod
COR – City of Roseville	mgd – million gallons per day	SOV – Schedule of Values
COS – City of Sacramento	mi – mile	SSWD – Sacramento Suburban Water District
Demobe – demobilization	Mobe – mobilization	SY – square yard
GW – groundwater	NMWC – Natomas Mutual Water Company	UOM – unit of measurement
hp – horsepower	PCWA – Placer County Water Agency	WTP – Water Treatment Plant
lf – linear foot	RM – River Mile	xx – crossing

CHAPTER 8 PERMIT REQUIREMENTS

To comply with Federal, State, and local laws, ordinances, and regulations, the SRWRS would be required to conduct or obtain numerous investigations, consultations, and permits. A Permit Acquisition Plan has been developed, and submitted separately, which discusses the permits that would need to be obtained after certification and approval of the EIS/EIR for the project. Discussed herein are some highlights from this plan.

8.1. REGULATORY REQUIREMENTS

The specific regulatory requirements have been organized by the three facilities of the SRWRS Elverta Diversion Alternative: Elverta Intake, North Natomas WTP, and pipelines. **Tables 8-1** through **8-3** summarize the permits or consultations required for each facility. These tables identify the regulation, the permit or consultation required, the permitting agency and contact information, and include some general notes.

Each consultation and permit has specific submittal requirements, as identified herein, and therefore would have different timing requirements for initiation with the permitting agency as well as final application submittal. **Table 8-4** provides an overview of the type of documentation typically submitted with the major permit applications.

8.2. RECOMMENDED TIMING OF PERMIT ACQUISITION

Many Federal and State permitting agencies have mandated periods for responding to permit applications. Using the documentation from **Table 8-4**, mandated response periods, and historical experience in obtaining permits, the timing for permit initiation and application periods has been estimated and is presented below.

8.2.1. Work to Be Completed During Preliminary Design Phase of the Project

As part of the preliminary design phase of the work, consultation would be initiated with numerous permitting agencies to begin discussion of project-specific conditions and design criteria that would need to be included in the design of ultimate facilities to obtain permits from these agencies. These contacts would not result in permits, but rather would identify the conditions and requirements for permit applications to be submitted as part of the final design when more detailed engineering design is available. This would include coordination with the following agencies:

- USACE (Section 404/10 Permit)
- DHS (Water Supply Permit)
- California Department of Transportation (Encroachment Permit)
- The Reclamation Board (Encroachment Permit)
- CVRWQCB (National Pollutant Discharge Elimination System (NPDES) Permit)
- Sacramento and Placer Counties (Encroachment Permits)
- Cities of Sacramento and Roseville (Encroachment Permits)

In addition to these consultations, several other permits and consultations can be completed or obtained during this phase of work, including the following:

- USCG (Aid to Navigation)
- FAA/Sacramento County Airport Service (Form 7460-1)
- UPRR (Encroachment Permit)
- Cal-OSHA (Gas Classifications)
- SAFCA (Flood Impact Consult)
- Reclamation District 1000 (Flood Impact Consult)
- CSD-1/Sacramento County Department of Water Resources (Sewer/Storm Drain Connection)
- Sacramento County (General Use and Building Permits)

8.2.2. Work to Be Completed During Final Design

As part of the final design, permit applications would be prepared for the agencies that were only consulted during the enhanced engineering analysis. This would include coordination with the following:

- USACE (Section 404/10 Permit)
- DHS (Water Supply Permit)
- California Department of Transportation (Encroachment Permit)
- The Reclamation Board (Encroachment Permit)
- CVRWQCB (NPDES Permit)
- Sacramento and Placer Counties (Encroachment Permits)
- Cities of Sacramento and Roseville (Encroachment Permits)

In addition to the permits above, several other permits and consultations would be ready to be completed or obtained during the final design, including the following:

- CDFG (Streambed Alteration Agreement)
- California State Lands Commission (Letter for Avoid Land Use Lease)
- CVRWQCB (Section 401 Water Quality Certification)
- SWRCB (Notice of Intent (NOI) for Stormwater and Low Threat Discharges)
- Sacramento Metropolitan Air Quality Control District (Generator Permit)
- PCACD (Generator Permit)
- Sacramento County (Tree Removal Permit)
- Placer County (Tree Removal Permit)

Table 8-1 Elverta Intake Permit Requirements

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
FEDERAL						
Federal Clean Water Act	Section 404 Individual Permit	U.S. Army Corps of Engineers	Mike Finnan	1325 J Street Sacramento, CA 95814-2922	(916) 557-5324	Need to conduct pre-application consultation and then complete and submit an Application for a Department of the Army Permit.
Rivers and Harbors Act	Section 10 Individual Permit	U.S. Army Corps of Engineers	Mike Finnan	1325 J Street Sacramento, CA 95814-2922	(916) 557-5324	Need to conduct pre-application consultation and then complete and submit an Application for a Department of the Army Permit.
	Private Aid to Navigation	U.S. Coast Guard	Brian Aldridge	MSO San Francisco Bay Waterways Management Bldg. 14 Coast Guard Island Alameda, CA 94501-5100	(510) 437-2983	Need to submit application for temporary and permanent aids to navigation and provide notice in Local Notice to Mariners during construction.
	Consultation for Airport Impacts	Federal Aviation Administration	Western Pacific Regional Office-Margie Drilling	Air-Traffic Division AWP 520 15000 Aviation Blvd. Hawthorne, CA 90260	(310) 725-3618 or (310) 725-3608 -General line	Need to submit Form 7460-1 to FAA in conjunction with Sacramento County Airport Service.
STATE						
	Aids to Navigation	California Department of Boating and Waterways	Mike Sotelo	2000 Evergreen Street, Ste. 100 Sacramento, CA 95815-3888	(916) 263-0787	Need to ensure that USCG private aids to navigation also meet State standards.
Fish and Game Code	Streambed Alteration Agreement	California Department of Fish and Game		Region 2 1701 Nimbus Rd, Ste A Rancho Cordova, CA 95670	(916) 358-2900/ (916) 445-0045	Need to submit application for streambed alteration.
California Health and Safety Code	Public Water System Permit	California Department of Health Services	Brian Kinney	DDWEM, Sacramento District P.O. Box 942732 Sacramento, CA 94234-7320	(916) 449-5688 (916) 449-5656	Need to meet with DHS to present design and obtain consensus on design criteria, then amend water supply permits.
California Code of Regulations and Public Resources Code	Land Use Lease	California State Lands Commission	Lorna Burkes	Land Mgmt. Division 100 Howe Avenue, Ste. 100S Sacramento, CA 95825-8202	(916) 574-1900	Need to obtain land use lease for intake located within riverbed. Not required if obtaining permit from USACE or The Reclamation Board.
Clean Water Act	Section 401 Water Quality Certificate	Central Valley Regional Water Quality Control Board	Patrick Gillum	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4709	Need to obtain The Water Quality Certification Waiver for USACE Permit.
Clean Water Act	Dewatering Permit - General Order No. 5-00-175	Central Valley Regional Water Quality Control Board	Michael Negrette	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4662	Need to obtain dewater permit for low-threat discharges for dewatering cofferdam at intake structure.
Clean Water Act	NPDES Permit	Central Valley Regional Water Quality Control Board	Jacque Kelley	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4764	May need to obtain permit to discharge stormwater to groundwater via detention basin or to surface water via discharge.
Clean Water Act	Stormwater Permit for Construction Activities	State Water Resources Control Board		P.O. Box 1977 Sacramento, CA 95812-1977	(916) 341-5537	Need to submit NOI for General Permit for Construction Activities.
California Streets and Highways Code	Encroachment Easement	California Department of Transportation	Rich Jones	703 B Street P.O. Box 911 Marysville, CA 95901	(530) 741-5374	Need permit to cross and potentially redesign/realign the Garden Highway.
	Tunneling Permit – Gas Classification	California Occupational Safety and Health Administration	Gerald Fulhgrum	Cal-OSHA - Division of Mining and Tunneling 2211 Park Towne Circle Sacramento, CA 95825-0414	(916) 574-2540	Only required if tunnel through levee.
California Water Code	Encroachment Permit	The Reclamation Board	Stephen Bradley	Floodway Prot. Section 1416 9th Street, Rm. 1623 Sacramento, CA 95814	(916) 574-0608/ (916) 574-0682	Need to submit application to encroach on floodway of the Central Valley.

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Table 8-1 Elverta Intake Permit Requirements (cont.)

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
LOCAL						
	Review Impacts to Levees	Sacramento Area Flood Control Agency	Pete Ghelfi	1007 7th Street, 7th Floor Sacramento, CA 95814	(916) 874-7606/ (916) 874-8289	Need Endorsement of Project by The Reclamation Board and USACE as well as coordinate with local projects for District 2
	Review Impact to Levees	Reclamation District 1000	Jim Clifton	1633 Garden Highway Sacramento, CA 95833	(916) 922-1449 (916) 922-9173	Need Endorsement of Project by The Reclamation Board
Clean Air Act and California HSC Section 42300	Permit to Construct and Operate Stationary Generators and Motorized Equipment	Sacramento Metropolitan Air Quality Management District	Brian Krebbs	777 12th Street, 3rd Floor Sacramento, CA 95814	(916) 874-4800	Need to submit application for intake pumps and motors as well as standby generator if used.
FAA Coordination	Consultation with Sacramento County Airport Service to Design Facilities to Meet Safety Standards and Presentation to FAA	Sacramento County Airport Service	Leonard Takayama/ Greg Rowe		(916) 874-0619/ (916) 874-0698	Need to meet all safety requirements for future Approach/Departure Zone of SMF.
County Zoning Ordinance	Use Permit	Sacramento County Department of Planning and Community Development	Charlie Dyer	827 7th Street, Rm. 230 Sacramento, CA 95814	(916) 874-6221 (information) (916) 874-6141/ (916) 874-6400	Conduct pre-application consultation and then submit application.
	Review Impact to Garden Highway	Sacramento County Department of Transportation	LDSIR-Tech Resources: Norm Novak	827 7th Street, Rm. 102 Sacramento, CA 95814	(916) 874-6544 (Tech. Res.) (916) 874-6873	Need to coordinate with CalTrans.
	Tree Pruning and Removal Permit	Sacramento County Public Works Agency	Technical Resources Section-Landscape Design and Tree Section: Henry Yasui	827 7th Street, Rm. 102 Sacramento, CA 95814	(916) 874-5278 Yasui Direct: (916) 874-8114/ (916) 874-1677	Need to submit application to remove riparian trees on river-side of levee.

Key:
Cal-OSHA – California Occupational Safety and Health Administration
DHS – Department of Health Services
FAA – Federal Aviation Administration
NOI – Notice of Intent
NPDES – National Pollutant Discharge Elimination System
SMF – Sacramento International Airport
USACE – United States Army Corps of Engineers
USG – United States Coast Guard

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Table 8-2 North Natomas WTP Permit Requirements

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
FEDERAL						
	Consultation for Airport Impacts	Federal Aviation Administration	Western Pacific Regional Office Margie Drilling	Air-Traffic Division AWP 520 15000 Aviation Blvd. Hawthorne, CA 90260	(310) 725-3618 or (310) 725-3608 General line	Need to submit Form 7460-1 to FAA in conjunction with Sacramento County Airport Service if located within Overflight Zone of SMF.
STATE						
California Health and Safety Code	Public Water System Permit	California Department of Health Services	Brian Kinney	DDWEM, Sacramento District P.O. Box 942732 Sacramento, CA 94234-7320	(916) 449-5688 (916) 449-5656	Need to meet with DHS to present design and obtain consensus on process selection and design criteria, then amend water supply permits.
Clean Water Act	Dewatering Permit - General Order No. 5-00-175	Central Valley Regional Water Quality Control Board	Michael Negrette	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4709 (916) 464-4662	May need to obtain dewater permit for low-threat discharges for construction-related dewatering.
Clean Water Act	NPDES Permit	Central Valley Regional Water Quality Control Board	Jacque Kelley	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4764	May need to obtain permit to discharge stormwater to groundwater via detention basin or to surface water via discharge.
Clean Water Act	Stormwater Permit for Construction Activities	State Water Resources Control Board		P.O. Box 1977 Sacramento, CA 95812-1977	(916) 341-5537	Need to submit NOI for General Permit for Construction Activities.
LOCAL						
Clean Air Act and California HSC Section 42300	Permit to Construct and Operate Stationary Generators and Motorized Equipment	Sacramento Metropolitan Air Quality Management District		777 12th Street, 3rd Floor Sacramento, CA 95814	(916) 874-4800	Need to submit application for standby generator if used.
FAA Coordination	Consultation with Sacramento County Airport Service to Design Facilities to Meet Safety Standards and Presentation to FAA	Sacramento County Airport Service	Leonard Takayama/ Greg Rowe		(916) 874-0619/ (916) 874-0698	Coordinate with Sacramento County Airport Service to submit information to FAA if located within Overflight Zone of SMF.
County Zoning Ordinance	Use Permit	Sacramento County Department of Planning and Community Development		827 7th Street Rm. 230 Sacramento, CA 95814	(916) 874-6221	Conduct pre-application consultation and then submit application.
	Building Permit	Sacramento County Department of Engineering and Administration	Bill Durkee	827 7th Street Rm 304 Sacramento, CA 95814	(916) 874-1691/ (916) 874-7100 Durkee Direct: (916) 874-6521/ (916) 874-5919	Need to determine if Building Permit required for Operations Building.
	Tree Pruning and Removal Permit	Sacramento County Public Works Agency	Technical Resources Section	827 7th Street Rm 102 Sacramento, CA 95814	(916) 874-5278	Need to obtain permit to remove trees if necessary.
	Storm Drain System Connection	Sacramento County Department of Water Resources	Kerry Schmitz	827 7th Street Rm 301 Sacramento, CA 95814	(916) 874-6851	Consultation to determine if storm drain system can be extended for connection.
	Collection System Connection	County Sanitation District 1/ Sacramento Regional County Sanitation District		10545 Armstrong Avenue Mather, CA 95655	(916) 876-6000	Consultation to Expand Services to New WTP for Wastewater.
	Septic System Permit	Sacramento County Environmental Management Department	Steve Kalvelage	8475 Jackson Rd., Ste 240 Sacramento, CA 95826	(916) 875-8484 Kalvelage Direct: (916) 875-8416/ (916) 875-8513	Consultation to install septic system if not able to extend SRCSD collection system.

Key:
DHS – Department of Health Services
FAA – Federal Aviation Administration
NOI – Notice of Intent
NPDES – National Pollutant Discharge Elimination System
SMF – Sacramento International Airport
SRCSD – Sacramento Regional County Sanitation District
WTP – Water Treatment Plant

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Table 8-3 Pipeline Permit Requirements

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
FEDERAL						
Federal Clean Water Act	Section 404 Individual Permit	U.S. Army Corps of Engineers	Mike Finnan	1325 J Street Sacramento, CA 95814-2922	(916) 557-5324	Need to conduct pre-application consultation and then complete and submit an Application for a Department of the Army Permit for Dry Creek and Pleasant Grove Creek crossings.
	Consultation for Airport Impacts	Federal Aviation Administration	Western Pacific Regional Office Margie Drilling	Air-Traffic Division AWP 520 15000 Aviation Blvd. Hawthorne, CA 90260	(310) 725-3618 or (310) 725-3608 General line	Need to submit Form 7460-1 to FAA in conjunction with Sacramento County Airport Service.
	Permit to Cross or Encroach	Union Pacific Railroad	Jon Devish	1800 Farnam Omaha, NE 68102	(402) 997-3563 (402) 997-3601	Need to submit application and Exhibit A for each crossing or encroachment.
STATE						
Fish and Game Code Section 1601	Streambed Alteration Agreement	California Department of Fish and Game		Region 2 1701 Nimbus Rd., Ste. A Rancho Cordova, CA 95670	(916) 358-2900/ (916) 445-0045	Need to submit application for streambed alteration of Dry Creek and Pleasant Grove Creek.
California Health and Safety Code	Public Water System Permit	California Department of Health Services	Brian Kinney	DDWEM, Sacramento District P.O. Box 942732 Sacramento, CA 94234-7320	(916) 449-5688 (916) 449-5656	Need to meet with DHS to present design and obtain consensus on design criteria, then amend water supply permits.
California Streets and Highways Code	Encroachment Easement	California Department of Transportation	Rich Jones	703 B Street P.O. Box 911 Marysville, CA 95901	(530) 741-5374	Need permit to cross Highway 99.
	Tunneling Permit – Gas Classification	California Occupational Safety and Health Administration	Gerald Fulhgrum	Cal-OSHA - Division of Mining and Tunneling 2211 Park Towne Circle Sacramento, CA 95825-0414	(916) 574-2540	Need Gas Classifications for potential tunnel/boring under roads and creeks.
Clean Water Act	Section 401 Water Quality Certificate	Central Valley Regional Water Quality Control Board	Patrick Gillum	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4709	Need to obtain Water Quality Certification Waiver for USACE Permit.
Clean Water Act	Dewatering Permit - General Order No. 5-00-175	Central Valley Regional Water Quality Control Board	Michael Negrette	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4662	May need to obtain dewater permit for low threat discharges for construction dewatering.
Clean Water Act	Stormwater Permit for Construction Activities	State Water Resources Control Board		P.O. Box 1977 Sacramento, CA 95812-1977	(916) 341-5537	Need to submit NOI for General Permit for Construction Activities.

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Table 8-3 Pipeline Permit Requirements (cont.)

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
LOCAL						
Clean Air Act and California HSC Section 42300	Permit to Construct and Operate Stationary Generators and Motorized Equipment	Placer County Air Control District	Zach Lee	11464 B Avenue DeWitt Center Auburn, CA 95603	(530) 889-7127/ (530) 889-7107	Need to submit application for Booster Pump Station generator if used.
FAA Coordination	Consultation with Sacramento County Airport Service to Design Facilities to Meet Safety Standards and Presentation to FAA	Sacramento County Airport Service	Leonard Takayama/ Greg Rowe		(916) 874-0619/ (916) 874-0698	Need to meet all safety requirements for current Overflight Zone and future Approach/Departure Zone at SMF.
	Tree Pruning and Removal Permit	Sacramento County Public Works Agency	Technical Resources Section		(916) 874-5278	Need to submit application to remove riparian trees on river-side of Sacramento River levee.
	Encroachment Permit	Sacramento County Department of Transportation (point of contact according to Web site)	Dennis Nakagawa	827 7th Street Rm. 102 Sacramento, CA 95814	(916) 874-5823 LDSIR: (916) 874-6544	Need permit to construct pipeline in road right-of-way.
	Encroachment Permit	Placer County Department of Public Works, Road Maintenance Division	Bob Vrooman	11444 B Avenue DeWitt Center Auburn, CA 95602	(530) 889-7565/ (530) 889-3528	Need permit to construct pipeline in road right-of-way.
	Encroachment Permit	City of Sacramento Department of Public Works	George Wilson	660 J Street Suite 250 Sacramento, CA 95814	(916) 808-1981/ (916) 448-8450	Need permit to construct pipeline in road right-of-way.
	Encroachment Permit	City of Roseville Public Works	Chris Kraft	311 Vernon Street Roseville, CA 95678	(916) 746-1300/ (916) 774-5379	Need permit to construct pipeline in road right-of-way.
	Tree Permit	Placer County Planning Department		11444 B Avenue DeWitt Center Auburn, CA 95602	(530) 886-3000	

Key:
Cal-OSHA – California Occupational Safety and Health Administration
DHS – Department of Health Services
FAA – Federal Aviation Administration
NOI – Notice of Intent
SMF – Sacramento International Airport
USACE – United States Army Corps of Engineers

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Table 8-4 Overview of Permit Documentation Needs

Permitting Agency/Permit	Documentation Type									
	Project Location/Map	Project Purpose	Project Description	Detailed Engineering	General Engineering	Environmental Assessment	Geotechnical Review	Drawings	Adjacent Property Holders	Other Permits/ Certifications
USACE – Section 404/10	√	√	√	√		√		√	√	√
USCG – Private Aid to Navigation	√		√	√				√		
FAA/Sac Co. Airport Service – Form 7460-1	√	√	√	√				√		
Union Pacific RR – Permit to Cross or Encroach	√		√	√			√	√		
CDFG – SAA	√		√	√		√		√		√
DHS – Water Supply Permit	√	√	√	√		√		√		√
CVRWQCB – NOI Dewater	√		√		√	√	√			√
CVRWQCB – Section 401 WQ	√	√	√		√	√		√	√	√
CVRWQCB – NPDES for Stormwater	√	√	√		√	√	√	√	√	√
SWRCB – NOI Construction Stormwater	√		√	√		√				√
CalTrans – Encroachment Permit	√		√	√			√	√		
Cal-OSHA – Gas Classification	√		√		√		√			
The Reclamation Board – Encroachment Permit	√	√	√	√		√	√	√		√
SMAQMD – Generator Permit	√			√						
PCACD – Generator Permit	√			√						
Sac Co. Planning Dept. – Use Permit	√	√	√		√	√		√	√	√
Sac Co. PW – Tree Removal Permit	√			√		√		√		
Sac Co. Eng./Admin. – Building Permit	√			√				√		
Sac Co. Water Resources – Storm Drain Connection	√		√		√	√	√			
CSD-1/SRCSD – Sewer Connection	√		√		√	√	√			
Sac Co. Env. Mgmt. Dept. – Septic System Permit	√			√		√	√	√		
Sac Co. Dept. of Transportation – Encroachment Permit	√			√				√		
Placer County PW – Encroachment Permit	√			√				√		
City of Sac PW – Encroachment Permit	√			√			√	√		
City of Roseville PW – Encroachment Permit	√			√				√		
Placer County Planning Department – Tree Permit	√		√	√		√		√		

Key:
Cal-OSHA – California Occupational Safety and Health Administration
CDFG – California Department of Fish and Game
CDHS – California Department of Health Services
CSD-1 – County Sanitation District 1
CVRWQCG – Central Valley Regional Water Quality Control Board
FAA – Federal Aviation Administration
NOI – Notice of Intent
NPDES – National Pollutant Discharge Elimination System
PCACD – Placer County Air Control District
PW – Public Works
Rec. Board – State of California Water Resources Agency – The Reclamation Board
RR – railroad
SAA – Streambed Alteration Agreement
Sac. Co. – Sacramento County
SMAQMD – Sacramento Metropolitan Air Quality Management District
SMF – Sacramento International Airport
SRCSD – Sacramento Regional County Sanitation District
USACE – United States Army Corps of Engineers
USCG – United States Coast Guard
WQ – Water Quality

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APPENDIX A

SUMMARY OF CONTAMINANTS

APPENDIX A
SRWRS ELVERTA DIVERSION ALTERNATIVE

Summary of Contaminants Currently Regulated by USEPA and DHS

Contaminant		Regulation	MCL (mg/L)
Inorganics (Section 64432)			
	Aluminum	DHS	1
	Antimony	Phase V	0.006
	Arsenic	NPDWR	0.010
	Barium	DHS/Phase II	1.0/2.0
	Beryllium	Phase V	0.004
	Cadmium	Phase II	0.005
	Chromium	DHS/Phase II	0.05/0.1
	Copper	LCR	1.3 [1,2]
	Cyanide	Phase V	0.15
	Fluoride	DHS/NPDWR	2.0/4.0
	Lead	LCR	0.015 [1,2]
	Mercury	Phase II	0.002
	Nickel	Phase V	0.1 [3]
	Selenium	Phase II	0.05
	Thallium	Phase V	0.002
Nitrate, Nitrite (Section 64432.1)			
	Nitrate	Phase II	10 as N (45 as NO3)
	Nitrite	Phase II	1 as N
	Nitrate + Nitrite	Phase II	10 (sum as N)
Asbestos (Section 64432.2)			
	Asbestos	Phase II	7 MFL (>10um)
Secondary Standards (Section 64449, Table 64449-A)			
	Aluminum	DHS	0.2
	Color	DHS	15 Units
	Copper	LCR	1
	Corrosivity	DHS	non-corrosive
	Foaming Agents	DHS	0.5
	Iron	DHS	0.3
	Manganese	DHS	0.05 (0.5 [1])
	Methyl-tert-butyl-ether (MTBE)	DHS	0.005
	Odor-Threshold	DHS	3 Units
	Silver	DHS	0.1
	Thiobencarb	DHS	0.001
	Turbidity	DHS	5 NTU
	Zinc	DHS	5

Summary of Contaminants
Currently Regulated by USEPA and DHS (contd.)

Contaminant		Regulation	MCL (mg/L)
Secondary Standards (Section 64449, Table 64449-B)			
	Total Dissolved Solids	DHS	500/1,000/1,500 [4]
	Specific Conductance	DHS	900/1,600/2,200 [4]
	Chloride	DHS	250/500/600 [4]
	Sulfate	DHS	250/500/600 [4]
General Mineral (Section 64449 (c) (2))			
	Bicarbonate	DHS	MO
	Carbonate	DHS	MO
	Hydroxide	DHS	MO
	Alkalinity	DHS	MO
	pH	DHS	MO
	Calcium	DHS	MO
	Magnesium	DHS	MO
	Sodium	DHS	MO
	Hardness	DHS	MO
(Volatile) Organic Chemicals (Section 64444, Table 64444-A (a))			
	Benzene	DHS/Phase I	0.001/0.005
	Carbon Tetrachloride	DHS/Phase I	0.0005/0.005
	o-Dichlorobenzene	Phase II	0.6
	p-Dichlorobenzene	DHS/Phase I	0.005/0.0785
	1,1-Dichloroethane	DHS	0.005
	1,2-Dichloroethane	DHS/Phase I	0.0005/0.005
	1,1-Dichloroethylene	DHS/Phase I	0.006/0.007
	cis-1,2-Dichloroethylene	DHS/Phase II	0.006/0.1
	trans-1,2-Dichloroethylene	DHS/Phase II	0.010/0.1
	Dichloromethane (Methylene chloride)	Phase V	0.005
	1,2-Dichloropropane	Phase II	0.005
	1,3-Dichloropropene	DHS	0.0005
	Ethylbenzene	Phase II	0.3
	Methyl-tert-butyl ether (MTBE)	DHS	0.013
	Monochlorobenzene	DHS/Phase II	0.07/0.1
	Styrene	Phase II	0.1
	1,1,2,2-Tetrachloroethane	DHS	0.001
	Tetrachloroethylene	Phase II	0.005
	Toluene	DHS/Phase II	0.15/1.0
	1,2,4-Trichlorobenzene	Phase V	0.005
	1,1,1-Trichloroethane	Phase I	0.2
	1,1,2-Trichloroethane	Phase V	0.005
	Trichloroethylene	Phase I	0.005

Summary of Contaminants Currently Regulated by USEPA and DHS (contd.)

Contaminant		Regulation	MCL (mg/L)
	Trichlorofluoromethane	DHS	0.15
	1,1,2-Trichloro-1,2,2-Triflouroethane	DHS	1.2
	Vinyl Chloride	DHS/Phase I	0.0005/0.002
	Xylenes (total)	DHS/Phase II	1.75/10
(Non-Volatile Synthetic) Organic Chemicals (Section 64444, Table 64444-A (b))			
	Acrylamide	Phase II	TT (PAP)
	Alachlor	Phase II	0.002
	Atrazine	Phase II	0.001
	Bentazon	DHS	0.018
	Benzo(a)pyrene	Phase V	0.0002
	Carbofuran	DHS/Phase II	0.018/0.04
	Chlordane	DHS/Phase II	0.0001/0.002
	2,4,-D	Phase II	0.07
	Dalapon	Phase V	0.2
	Dibromochloropropane	Phase II	0.0002
	Di (2-ethylhexyl) Adipate	Phase V	0.4
	Di (2-ethylhexyl) Phthalate	DHS/Phase V	0.004/0.006
	Dinoseb	Phase V	0.007
	Diquat	Phase V	0.02
	Endothall	Phase V	0.1
	Endrin	Phase V	0.002
	Epichlorohydrin	Phase II	TT (PAP)
	Ethylene Dibromide	Phase II	0.00005
	Glyphosate	Phase V	0.7
	Heptachlor	DHS/Phase II	0.00001/0.0004
	Heptachlor Epoxide	DHS/Phase II	0.00001/0.0002
	Hexachlorobenzene	Phase V	0.001
	Hexachlorocyclopentadiene	Phase V	0.05
	Lindane	Phase II	0.0002
	Methoxychlor	Phase II	0.03
	Molinate	DHS	0.02
	Oxamyl (vydate)	Phase V	0.05
	Pentachlorophenol	Phase II	0.001
	Picloram	Phase V	0.5
	PCBs	Phase II	0.0005
	Simazine	Phase V	0.004
	Thiobencarb	DHS	0.07
	Toxaphene	Phase II	0.003
	2,3,7,8-TCDD (Dioxin)	Phase V	3.00E-08
	2,4,5-TP (Silvex)	Phase II	0.05

Summary of Contaminants
Currently Regulated by USEPA and DHS (contd.)

Contaminant		Regulation	MCL (mg/L)
Unregulated (Volatile) Organic Chemicals (Section 64450, Table 64450-A)			
	Dichlorodifluoromethane	DHS	1.0 [1]
	1,2,3-Trichloropropane	DHS	0.000005 [1]
	Ethyl-tert-butyl-ether (ETBE)	DHS	MO (if vulnerable)
	tert-Amyl-methyl ether (TAME)	DHS	MO (if vulnerable)
	Perchlorate	DHS	0.004 [1]
	Boron	DHS	1.0 [1]
	Hexavalent Chromium	DHS	MO (if vulnerable)
	tert-Butyl alcohol	DHS	0.012 [1]
	Vanadium	DHS	0.05 [1]
Natural Radioactivity (Section 64441)			
	Gross Alpha Particle Activity	NPDWR	15 pCi/L
	Combined Radium 226 & 228	NPDWR	5 pCi/L
	Uranium	DHS	20 pCi/L
Man-Made Radioactivity (Section 64443)			
	Tritium	DHS	20,000 pCi/L
	Strontium-90	DHS	8 pCi/L
	Gross Beta Particle Activity	NPDWR	50 pCi/L
Disinfection By-Products			
	Total Trihalomethanes (Chloroform, Bromoform, Chlorodibromomethane, Bromodichloromethane)	Stage 1 D/DBP Rule	0.08
	Haloacetic Acids 5 (Mono, di, and tri-chloroacetic acid, mono and di-bromoacetic acid)	Stage 1 D/DBP Rule	0.06
	Chlorite	Stage 1 D/DBP Rule	1
	Bromate	Stage 1 D/DBP Rule	0.01
Disinfection By-Product Precursors			
	Total Organic Carbon	Stage 1 D/DBP Rule	TT (percent Removal)
Disinfectants			
	Chlorine (as Cl2)	Stage 1 D/DBP Rule	4 [5]
	Chloramines (as Cl2)	Stage 1 D/DBP Rule	4 [5]
	Chlorine Dioxide (as ClO2)	Stage 1 D/DBP Rule	0.8 [5]
Microbial			
	Giardia Lamblia	SWTR	TT(3-log Reduction)
	Legionella	SWTR	TT
	Viruses	SWTR	TT(4-Log Reduction)
	Disinfectant Residual	SWTR	TT(detectable)
	Total Coliform	TCR	TT(<5percent mo. samples pos., if >40 samples per month)
	Fecal Coliform	TCR	TT (positive sample)
	E. Coli	TCR	TT (positive sample)

Summary of Contaminants Currently Regulated by USEPA and DHS (contd.)

	Contaminant	Regulation	MCL (mg/L)
	Turbidity	IESWTR	TT (<0.3 in 95percent CFE samples, <1 in 100percent CFE)
	Cryptosporidium	IESWTR	TT(2-log Reduction)
Additional Organics with Action Levels			**Action Levels**
	Aldicarb	DHS	0.007
	Aldrin	DHS	0.000002
	Baygon	DHS	0.03
	a-Benzenehexachloride	DHS	0.000015
	b-Benzenehexachloride	DHS	0.000025
	n-butylbenzene	DHS	0.26
	sec-butylbenzene	DHS	0.26
	tert-butylbenzene	DHS	0.26
	Captan	DHS	0.0015
	Carbaryl	DHS	0.7
	Carbon disulfide	DHS	0.16
	Chlorate	DHS	0.8
	Chloropicrin	DHS	0.056
	2-chlorotoluene	DHS	0.14
	4-chlorotoluene	DHS	0.14
	Chlorpropham	DHS	1.2
	1,3-Dichlorobenzene	DHS	0.6
	2,4-Dimethylphenol	DHS	0.1
	1,4-Dioxane	DHS	0.003
	Diazinon	DHS	0.006
	Dieldrin	DHS	0.000002
	Diphenamide	DHS	0.2
	Ethion	DHS	0.004
	Ethylene glycol	DHS	14
	Formaldehyde	DHS	0.1
	Isopropylbenzene	DHS	0.77
	Malathion	DHS	0.16
	Metam sodium	DHS	0.02
	Methyl isobutyl ketone (MIBK)	DHS	0.12
	Methylisothiocyanate	DHS	0.05
	Methyl parathion	DHS	0.002
	Napthalene	DHS	0.17
	N-Nitrosodimethylamine (NDMA)	DHS	0.00001
	Parathion	DHS	0.04
	Pentachloronitrobenzene	DHS	0.02
	Phenol	DHS	4.2
	n-propylbenzene	DHS	0.26
	Trithion	DHS	0.007

Summary of Contaminants
Currently Regulated by USEPA and DHS (contd.)

	Contaminant	Regulation	MCL (mg/L)
	2,3,5,6-tetrachloroterephthalate	DHS	3.5
	1,2,4-Trimethylbenzene	DHS	0.33
	1,3,5-Trimethylbenzene	DHS	0.33

Notes:
[1] - Action Level
[2] - Based on 90th Percentile of Tap Water Samples
[3] - DHS MCL lower than EPA, EPA remanded in 1995
[4] - Recommended/Upper/Short Term MCLs
[5] - Maximum Residual Disinfectant Level (MRDL)

Key:
CFE – Combined Filter Effluent
D/DBP – Disinfectants and Disinfection By-Products
DHS – California Department of Health Services
IESWTR – Interim Enhanced Surface Water Treatment Rule
LCR – Lead and Copper Rule
MCL – Maximum Contaminant Level
MO – Monitored Only
NPDWR – National Primary Drinking Water Regulation
PAP – Polymer Addition Practices
SWTR – Surface Water Treatment Rule
TCR – Total Coliform Rule
TT – Treatment Technology
USEPA – United States Environmental Protection Agency

APPENDIX B

ROSEVILLE CHEMICAL BOOSTER FEED FACILITY

APPENDIX B

ELVERTA DIVERSION ALTERNATIVE
ROSEVILLE CHEMICAL BOOSTER FEED FACILITY

Roseville delivers fluoridated water to its consumers; therefore a remote chemical feed facility would be required to fluoridate treated water from the Elverta WTP. Fluoridation typically depresses the pH; therefore, caustic soda would also be provided to increase the pH to meet Roseville's distribution system requirements. Finally, sodium hypochlorite would be provided to ensure that an adequate disinfectant residual is maintained. Roseville will confirm the location of this facility during future land-use planning efforts.

Roseville proposes to install feed and storage facilities for hydrofluosilicic acid, caustic soda, and sodium hypochlorite. Hydrofluosilicic acid is obtained as a liquid (23 percent solution) in bulk delivery. Storage and feed equipment would need to be constructed of specific materials to resist corrosion. Storage would need to be 100 percent contained for maximum acid volume. Space would be provided for 30 days of storage. Caustic soda is obtained as a liquid (25 percent solution) in bulk delivery. Space would be provided for 30 days of storage. Sodium hypochlorite would be used as the secondary disinfectant. It would be obtained as a liquid, delivered as a 12.5 percent solution. Thirty days of storage would be provided. **Table B-1** summarizes chemical feed and storage requirements. A preliminary floor plan and elevation are shown in **Figure B-1**.

Table B-1 Summary of Chemical Feed and Storage Requirements

Chemical	Storage Criteria	Storage Weight or Volume	Type of Container	Number of Containers
Caustic Soda	30 days @ 5 mg/L and 10 mgd	4,600 gallons	Horizontal steel	2 – 2,500 gallon
Hydrofluosilicic Acid	30 days @ 0.8 mg/L as F and 10 mgd	1,000 gallons	Fiberglass	2 – 500 gallon
Sodium Hypochlorite	30 days @ 0.5 mg/L as Cl and 10 mgd	1,900 gallons	Vertical steel	2 – 1,000 gallon

Key:
Cl – chlorine
F – fluoride
mg/L – milligrams per liter
mgd – million gallons per day

1.1. LOCATION OF FACILITIES

One potential site is a parcel of land located just south of Roseville's existing WTP. The site area is rectangular-shaped with a total land area of approximately 5 acres, including roadways. The site's facilities are arranged to simplify connection to the incoming and outgoing pipelines.

The chemical booster feed station would be located to the east of the treated water tanks. The location near the main entrance from Phillip Road would allow for more convenient chemical deliveries. Roseville or PCWA, requires a booster pump station for this location, space has been allocated and it has been assumed that it would be located at the southeast corner of the site. Distribution system piping could exit the site east onto Phillip Road.

1.2. OPERATING CHARACTERISTICS

The Roseville booster chemical feed facility would be operating continuously when Roseville is taking delivery of Elverta WTP treated water. All facilities, including storage and feed equipment, would be located inside the booster chemical feed facility building, which would minimize, if not eliminate, noise associated with the facility. The design would include elements such as equipment and instrumentation that would minimize the amount of operations and maintenance. No permanent staff would be located on site. The facility would require regular inspection, which is expected to be daily, to ensure that all facilities were secure and operating properly. These inspections are expected to be conducted by a single person.

Caustic soda, sodium hypochlorite, and hydrofluosilicic acid would be stored in bulk on site as described above. Chemical storage facilities have been sized to provide 30 days of chemicals on site for maximum daily flow; therefore, it is expected that chemical delivery trucks would be making at most, monthly deliveries. All bulk fluids would be delivered in transport trucks. The size of these trucks would vary by chemical type and delivery company. All chemical storage would be contained within the chemical feed and storage building, and Roseville's safety procedures and best management practices would be implemented at this facility.

1.3. CONSTRUCTION CHARACTERISTICS

Construction activities would involve small amounts of grading, and erecting the new chemical feed and storage building. It is expected that the excavated materials would be used, if acceptable from an engineering perspective, as fill on site. Standard construction methods are proposed.

Construction-related traffic (e.g., materials delivery trips, workers,) would access the site from Phillips Road. Materials trips would depend on geotechnical findings regarding the usability of soil for fill and scheduling of construction activities. A traffic control plan would be prepared by the contractor and reviewed by Roseville to make sure traffic is safely routed by the work site. No off-site facilities are proposed for this project.

Safety on the construction site would be the responsibility of the contractor. The contractor would have a company safety program and a job-specific safety program, administered by a project safety officer. Typical procedures would include weekly safety meetings with the construction crew and hazard analyses prepared before the beginning of each new operation. OSHA and Cal-OSHA standards would apply for all work.

The construction contract documents would include a general SWPPP. The construction contractor would be required to submit a specific, more detailed SWPPP. The general plan would outline minimum requirements that must be met to minimize erosion and control sediments. The general and specific SWPPPs would comply with the county sediment and erosion control ordinances. Typical best management practices that would be used include the following:

- Covering all exposed slopes and stockpiles with plastic, straw, or hydroseed
- Placing silt fences at the downstream side of all work areas
- Placing a sediment filter in each drop inlet
- Sweeping all work areas frequently
- Constructing sediment ponds in key locations

- Placing waddles or hay bales across steep, disrupted slopes
- Constructing gravel driveways at the work site exit

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APPENDIX C

DRAFT TECHNICAL MEMORANDUM ELKHORN/ELVERTA DIVERSION ALTERNATIVE

BASELINE ROAD VS. ELVERTA ROAD

PIPELINE PRELIMINARY ROUTING ANALYSIS

Draft Technical Memorandum

Elkhorn/Elverta Diversion Alternative

Baseline Road vs. Elverta Road Pipeline Preliminary Routing Analysis

October 2003

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

The purpose of this Technical Memorandum (TM) is to evaluate two alternative routes for a segment of the 72-inch pipeline to be installed as part of the proposed Elkhorn/Elverta Diversion Alternative, as defined in the Phase I Engineering Report[1]. The overall pipeline would convey treated water from the Elverta Road Water Treatment Plant (WTP), located on Elverta Road near Garden Highway (**Figure 1**), to Placer County Water Agency (PCWA), City of Roseville (Roseville), and Sacramento Suburban Water District (SSWD). The pipeline segment evaluated in this TM is the portion of pipeline between the intersection of Elverta Road and East Levee Road, and the intersection of Elverta Road and Watt Avenue.

PRELIMINARY ANALYSIS OF ROUTE ALTERNATIVES

The first alternative route would follow East Levee Road/Natomas Road, Riego Road/Baseline Road, and Watt Avenue (referred to as the Baseline Road alternative) and the second would follow Elverta Road and Watt Avenue (referred to as the Elverta Road alternative; see **Figure 1**). The Baseline Road alternative was chosen as the preferred route in the Phase 1 Engineering Report based on aerial photography, current street maps, existing information provided by the cost sharing partners, and field investigation. The approach used to determine the preferred alignment included avoiding encroachment into private property, following the most direct route on roadway or existing right-of-way (ROW), avoiding major disruption to the existing utilities, and, where possible, avoiding highly populated areas. However, at the August 19th, 2003 Sacramento River Water Reliability Study (SRWRS) Study Management Team (SMT) meeting, concerns were expressed about the feasibility and appropriateness of the selected route and routing the pipeline along Elverta Road was suggested. In order to address these concerns, existing conditions along both routes were evaluated according to the criteria listed below in **Table 1**. Environmental issues are not addressed in this evaluation. A complete environmental analysis of all project alternatives is being complete as a separate task and will be used in final routing determination.

Table 1. Route Analysis Criteria

Criteria	Description
Capital Cost	Total cost associated with design, construction, and labor
Right-of-Way (ROW)	Existing area available for construction, operation, and maintenance
Traffic Impacts	Traffic impacts expected during construction (e.g., road closures, lane reductions)
Population	Population density and distribution
Disruption of Utilities	Initial assessment of quantity and types of utilities expected to be encountered
Public Concerns	Factors that could result in negative impacts to the public (i.e., reduced access to residences or public facilities, noise, reduced access to business resulting in loss of income)

[1] MWH. 2003. Appendix C, Phase I Engineering Report, Sacramento River Water Reliability Study. September.

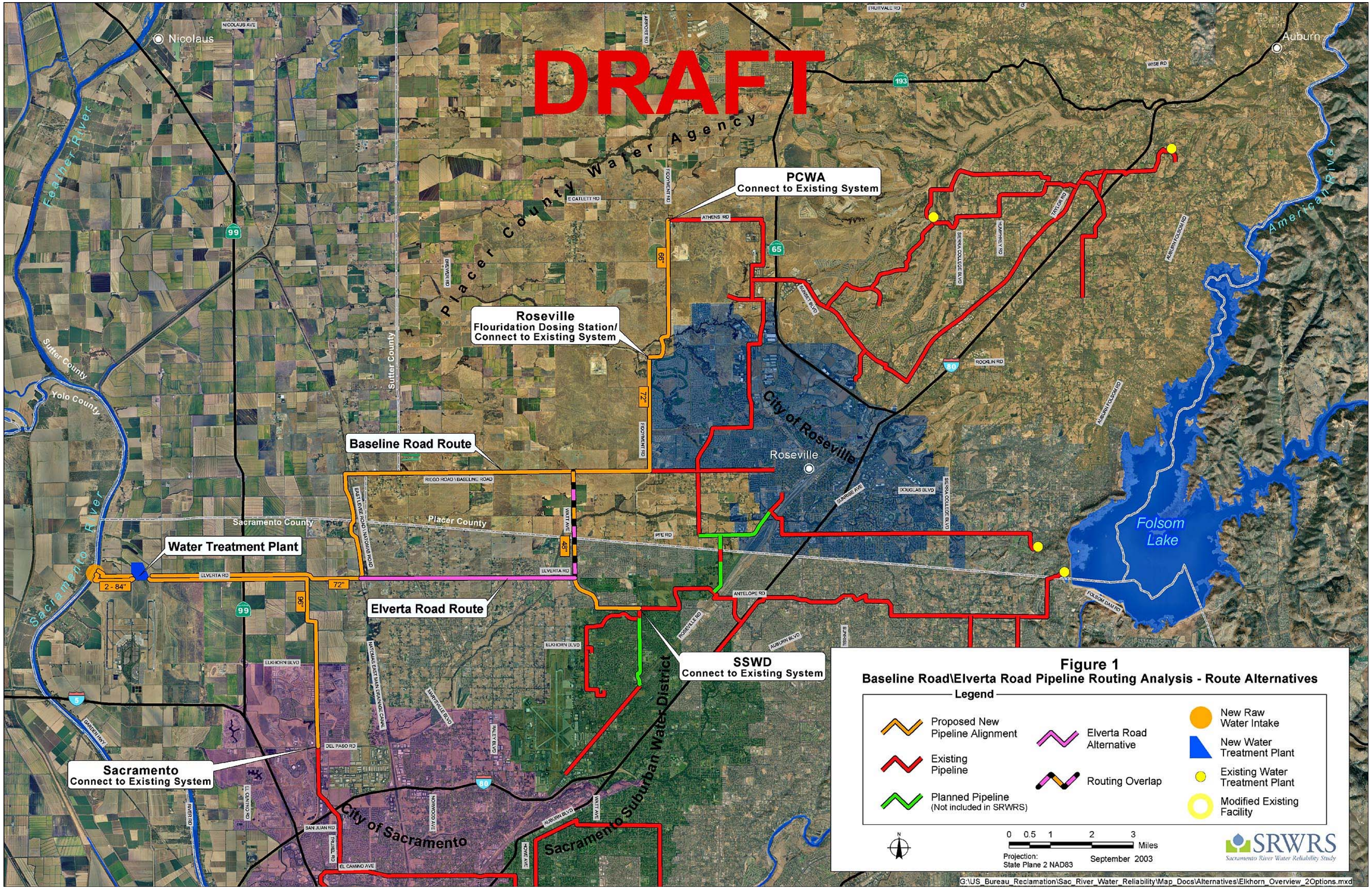


Figure 1. Baseline Road/Elverta Road Pipeline Routing Analysis – Route Alternatives

Baseline Road Alternative

For the Baseline Road alternative (**See Figure 1**), a 72-inch diameter pipeline would follow Elverta Road to the Natomas East Main Drainage canal, where it would turn north, and follow East Levee Road/Natomas Road to Riego Road where it would turn east, and continue east on Baseline Road (Riego Road changes name to Baseline Road when it leaves Sutter County and enters Placer County). At the intersection of Baseline Road and Watt Avenue, approximately 1.8 miles before reaching Fiddyment Road, a 48-inch pipeline would extend from the 72-inch pipeline and continue south along Watt Avenue to deliver 15 million gallons per day (mgd) to SSWD. The 72-inch pipeline would continue along Baseline Road to Fiddyment Road where it would deliver 75 mgd to Roseville and PCWA. An evaluation of the Baseline Road alternative with respect to the six route analysis criteria is presented in the following sections.

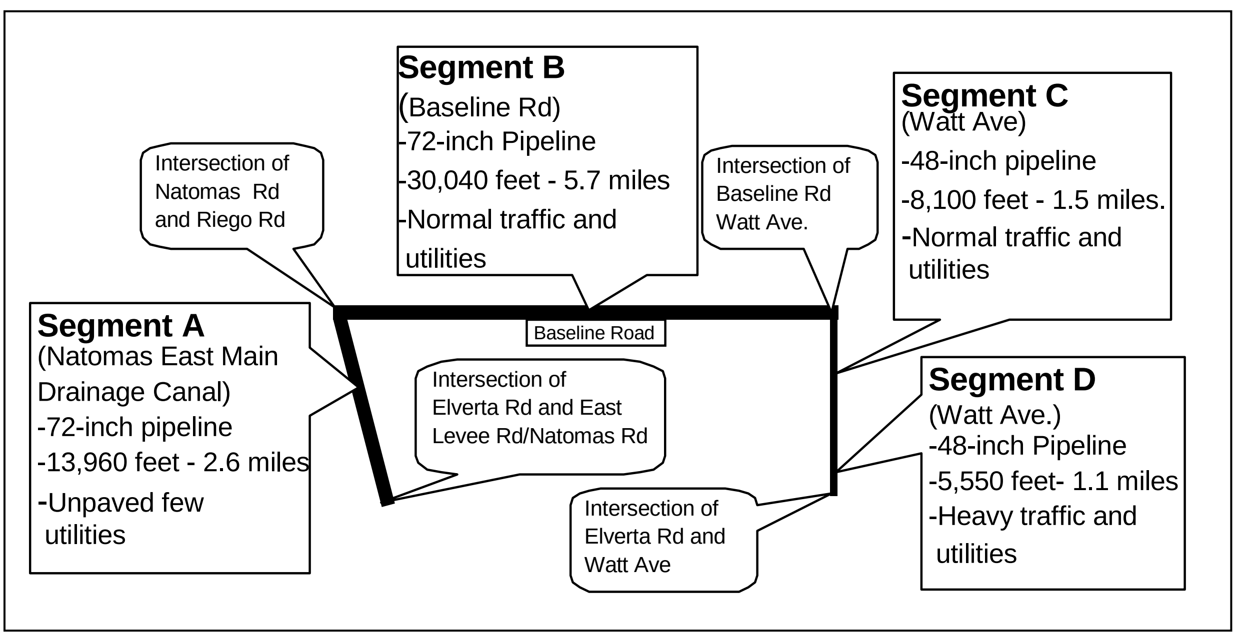
Capital Cost

This section develops cost estimates associated with installing the pipeline for the Baseline Road alternative. Estimates include costs for materials, installation, and labor. Cost estimates are based on a unit cost, as discussed below.

Pipeline Unit Costs

To determine the cost of constructing the conveyance pipeline, a unit cost was developed for each pipeline condition. Each pipeline segment with similar characteristics was classified: Segment A - unpaved with few utilities, Segments B and C - normal traffic and utilities, and Segment D - heavy traffic and utilities. **Figure 2** shows a schematic of the four segments of the Baseline Road alternative and the length of each segment. The corresponding unit cost for each pipeline segment is as follows: unpaved few utilities; $8/diameter-inch/LF, normal traffic and utilities; $10/diameter-inch/LF, and heavy traffic and utilities; $12/diameter-inch/LF. Estimates were considered accurate at the feasibility level of the SRWRS, and may range between 30 percent above and 20 percent below actual construction costs.

Figure 2. Segment Analysis for the Baseline Road Alternative



Tunneling Costs

For the Baseline Road alternative, the 48-inch pipeline crosses Dry Creek, a medium-sized stream. Therefore, in addition to costs for materials, costs would be incurred for tunneling. Tunneling costs were estimated based on other projects that involved creek crossings under similar conditions. Dry Creek is approximately 250 feet long; a distance of 150 feet was added to this length for installing launching and receiving shafts (50-feet-deep shafts were assumed for this crossing). The tunneling cost used for a 48-inch pipeline is of $27/Diameter-inch/LF.

Estimated Route Cost

The total cost for the Baseline Road Alternative is $58,593,000 as shown in **Table 2** below.

Table 2. Cost Estimate for Baseline Road Alternative

Description	Unit	Quantity	Cost/Unit	Extended Cost
PIPELINE - BASELINE ROAD ALTERNATIVE				
Segment A - 72-inch diameter	LF	13,960	$576	$8,040,960
Segment B - 72-inch diameter	LF	30,040	$720	$21,628,800
Segment C - 48-inch diameter	LF	8,100	$480	$3,888,000
Segment D - 48-inch diameter	LF	5,550	$576	$3,196,800
Subtotal				**$36,755,000**
TUNNELING - DRY CREEK ON WATT AVENUE				
Tunneling - 48-inch Pipeline	LF	400	$1,296	$518,500
Tunneling Mobilization	LS	1	n/a	$150,000
Tunneling Shafts[1]	CY	3000	$25	$75,000
Subtotal				**$743,500**
Baseline Road Alternative Cost				
Subtotal				**$37,499,000**
25% Engineering, Environmental, Administration and Legal Fees		25%		$9,375,000
Subtotal				**$46,874,000**
25% Contingency Fees	25%			$11,719,000
TOTAL CAPITAL COST				**$58,593,000**

[1] Cost Includes launching and receiving Shafts

Key:
cy cubic yard
ft feet
lf linear foot
n/a not applicable

Right-of-Way

The right-of-way for Baseline Road between East Levee Road and Fiddyment Road is 60 feet centered on the roadway centerline. The existing roadway between the East Levee Road/Natomas Road and Fiddyment Road is a two lane paved road approximately 30 feet wide. The current right-of-way/roadway configuration provides good construction access. Placer County Road Expansion and Improvements plans to widen Baseline Road to a four-lane road; the timeline for this expansion has not yet been determined.

The right-of-way for East Levee Road/Natomas Road and Watt Avenue have not yet been determined.

Traffic Impacts

East Levee Road is a low-use north/south road located in Sutter and Sacramento counties. The portion of East Levee Road associated with the Baseline Road route alternative borders the Steelhead Creek/Natomas East Main Drainage Canal. East Levee Road runs along the top of the levee. Initial plans for the alignment would involve placing the 72-inch pipeline adjacent to the west toe of the levee and backfilling this segment, therefore widening the overall levee section. Trenching activities would be minimal and traffic impacts during construction would be limited to temporary construction vehicles and reduced access to roadway shoulder.

Baseline Road is a moderate-to-high-use east/west corridor in Sutter and Placer counties. Traffic count information from the Placer County Department of Transportation is summarized in **Table 3**. Traffic control measures would be required to maintain an adequate flow of traffic. Traffic control measures such as nighttime construction could be required and would likely result in slower production rates for open cut construction.

Table 3. Seven-Day Traffic Counts for Baseline Road, March 2003

Date	Period of Count	Location	Results
March 2003	**7 days**	West of Watt Avenue	Eastbound lane: 5,153 Westbound lane:7,091 **Total: 12,244**
March 2003	**7 days**	East of Watt Avenue	Eastbound lane: 7,084 Westbound lane: 6,573 **Total: 13,657**

Source: Traffic count information provided by the Placer County Department of Transportation

Population Density

Population density along Baseline Road is low[2], as shown in **Figure 3**. The population to the south of Baseline Road is projected to increase through urban development, while land use to the north is currently zoned for agricultural use.

Disruption of Utilities

The extent of domestic utilities serving communities in nearby areas can be estimated in **Figure 3**. During an initial site visit, few utilities were observed along Baseline Road. Most of the utilities were found on Riego Road between Steelhead Creek/Natomas East Main Drainage Canal and the Placer County line, primarily confined to overhead lines (three-phase electrical and telephone). Water, gas and cable were not observed in the initial reconnaissance. The portion of the route extending farther east from Steelhead Creek/Natomas East Main Drainage Canal contained very few utilities.

Public Concerns

The key concern for the Baseline Road alternative would be local and through traffic disruption. Construction noise and dust would be of minimal concern due to the low density and development along this route.

[2] Information gathered from Census 2000, Population Data

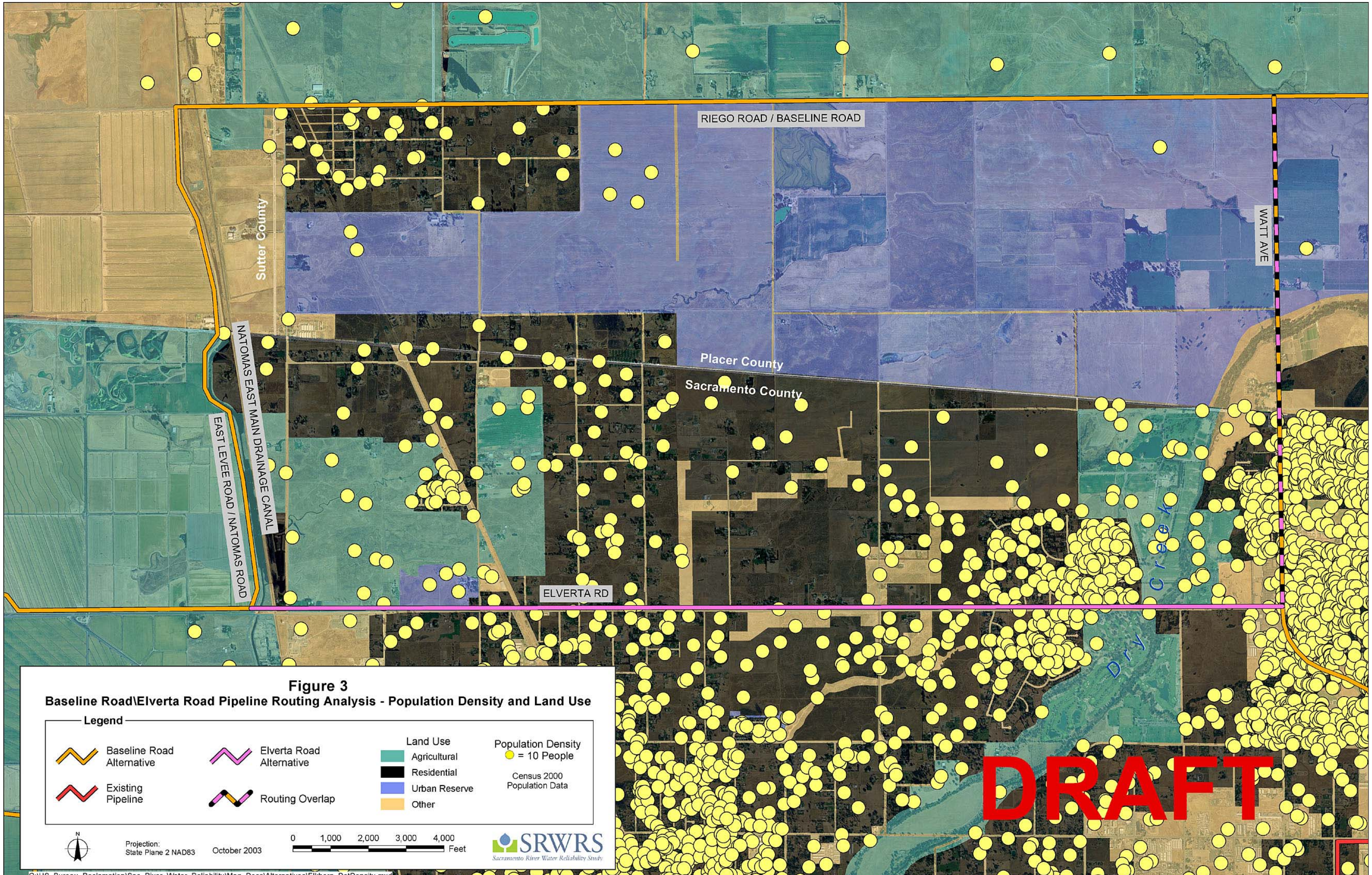


Figure 3. Baseline Road/Elverta Road Pipeline Routing Analysis – Population Density and Land Use

Elverta Road Alternative

For the Elverta Road alternative (**See Figure 1**) a 72-inch diameter pipeline would follow Elverta Road to Watt Avenue. From Watt Avenue the 72-inch pipeline would continue north on Watt Avenue where it would deliver 75 mgd to PCWA and Roseville, while a 48-inch pipeline would split off and continue south to deliver 15 mgd to SSWD. An evaluation of the Elverta Road alternative with respect to the six route analysis criteria is presented in the following sections.

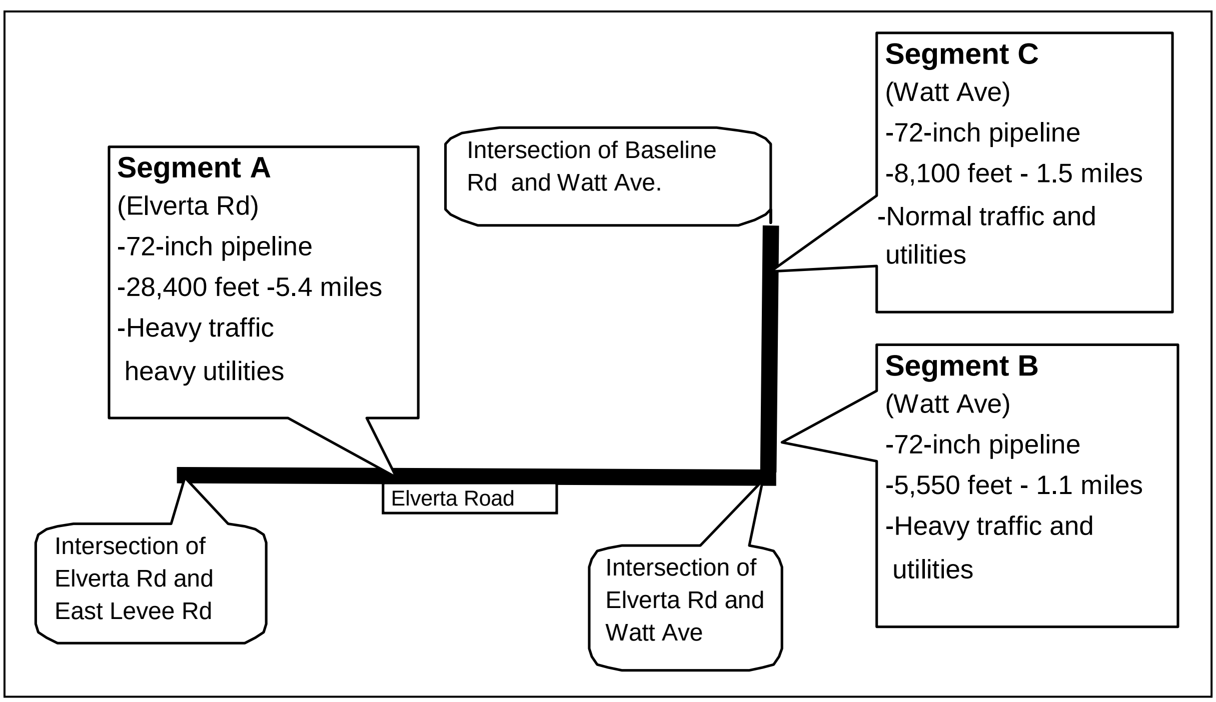
Capital Cost

This section develops cost estimates associated with installing the pipeline for the Elverta Road alternative. Estimates include costs for materials, installation, and labor. Cost estimates are based on unit costs, as discussed below.

Pipeline Unit Costs

To determine the cost of installing the conveyance pipeline, a unit cost was developed for each pipeline condition. Each pipeline segment with similar characteristics was classified: Segments A and B– heavy traffic and utilities, Segment C - normal traffic and utilities. **Figure 4** shows a schematic of the three segments of the Elverta Road alternative and the length of each segment. The corresponding unit cost for each pipeline segment is as follows: unpaved few utilities; $8/diameter-inch/LF, normal traffic and utilities; $10/diameter-inch/LF, and heavy traffic and utilities; $12/diameter-inch/LF. Estimates were considered accurate at the feasibility level of the SRWRS, and may range between 30 percent above and 20 percent below actual construction costs.

Figure 4. Segment Analysis for Elverta Road Alternative.



Tunneling Costs

For the Elverta Road alternative, the 72-inch pipeline would cross Dry Creek twice. Therefore, in addition to costs for materials, costs would be incurred for tunneling twice beneath this medium-sized stream. Tunneling costs estimated were based on other projects that involved creek crossings under similar conditions. Both crossings are approximately 250 feet long; a distance of 150 feet was added to each crossing for installing launching and receiving shafts. (A 50-feet-deep shaft is assumed for these crossings.) The tunneling cost for a 72-inch pipeline is of $38/Diameter-inch/LF.

Tunneling Costs

The total cost for the Elverta Road Alternative is $59,774,000 as shown in **Table 4** below.

Table 4. Cost Estimate for Elverta Road Alternative

Description	Unit	Quantity	Cost/Unit	Extended Cost
PIPELINE - ELVERTA ROAD ALTERNATIVE				
Segment A - 72-inch diameter	LF	28,400	$864	$24,538,000
Segment B - 72-inch diameter	LF	5,550	$864	$4,796,000
Segment C - 72-inch diameter	LF	8,100	$720	$5,832,000
Subtotal				**$35,166,000**
TUNNELING - DRY CREEK ON ELVERTA ROAD AND WATT AVENUE				
Tunneling - 72-inch Pipeline	LF	800	$2,736	$2,189,000
Tunneling Mobilization	LS	2	n/a	$600,000
Tunneling Shafts[1]	CY	12,000	$25	$300,000
Subtotal				**$3,089,000**
Elverta Road Alternative Cost Subtotal				**$38,255,000**
25% Engineering, Environmental, Administration and Legal Fees				$9,564,000
Subtotal				**$47,819,000**
25% Contingency Fees				$11,955,000
PIPELINE AND TUNNELING COST				**$59,774,000**

[1] Cost Includes launching and receiving Shafts

Key:
cy cubic yard
ft feet
lf linear foot
n/a not applicable

Right-of-Way

The right-of-way for Elverta Road between Watt Avenue and East Levee Road varies in width. The width of the right-of -way for Elverta Road is as follows: at 16th street the right-of-way is approximately 80 feet, at 28th street the right-of way is approximately 96 feet, at El Modena Road the right-of-way is approximately 50 feet, and at Rio Linda Blvd the right-of-way is approximately 45 feet. In the area east of El Modena Road through Rio Linda Blvd, homes are very close to the roadway and the narrow right-of-way. Sacramento County plans to expand this portion of the road to a four-lane road; long-term projections for road improvement include widening Elverta Road to six lanes.

The right-of-way for Watt Avenue has not yet been determined.

Traffic Impacts

Elverta Road is a heavily used east/west corridor in Sacramento County, and therefore has a significant volume of traffic, particularly on weekdays (**Table 5**). Traffic Control measures would be required to maintain an adequate flow of traffic during construction along all segments. Traffic control measures such as nighttime construction, would likely result in slower production rates, affecting the general public and cost.

A portion of Elverta road approximately 1.2 miles long, that extends between El Verano Road and Rio Linda Road, narrows to 28 feet in width to two lanes with limited space between the edge of the road and property lines. This particular section has many small residences (approximately 30 small homes). Traffic controls measures for this section would likely include re-routing or temporary road closure, which would likely have adverse impacts on the local community and commuters.

Table 5. 24-Hour Traffic Counts on Elverta Road, August 2003

Date	Period of Count	Location	Results
August 2003	**24 hours**	East of Watt Avenue	Eastbound lane: 9,100 Westbound lane: 9,448 **Total: 18,548**
August 2001	**24 hours**	Intersection with El Centro Road	**Total Count: 6,370**

Source: Traffic count information provided by the Sacramento County Department of Transportation

Population Density

Population distribution along Elverta Road is shown **Figure 3**.[3] Population density along Elverta Road is moderate on the west end of the segment of interest but increases approximately 1.5 miles west of Dry Creek. This area is mostly divided into small property parcels, population is expected to increase in this area.

Disruption of Utilities

During a site visit to the segment of the proposed route from the Natomas East Main Drainage Canal to Watt Avenue, areas with many utilities were observed, including a high voltage transformer station, overhead lines (three-phase and telephone) along the north and/or south sides of Elverta Road, and natural gas lines.

Public Concerns

There are many small residences on both sides of Elverta Road from El Verano Road to Rio Linda Road, an approximately 1-mile portion of the route. Approximately 30 homes within this area, with parcels shape narrow in the front and deep in length (65-feet by 475-feet in depth). The space between property lines and the edge of the road is approximately 10 to 15 feet. Placing a 72-inch-diameter pipeline and the required trench would impact every homeowner in the area with dust, construction noise, and traffic flow reductions. Many of these homes do not have alternate access; therefore, open cut trenching would restrict routine access to their properties

[3] The information on this figure was gathered from the 2000 population census

CONCLUSION

The two route alternatives are compared in **Table 6** with respect to each of the six criteria used in this evaluation.

Table 6. Comparison of Alternatives

Cost	The estimated costs of construction for the Baseline Road alternative ($58,593,000) and the Elverta Road alternative ($59,744,000) are comparable. The cost for Baseline Road is slightly lower than the Elverta Road mostly due to the costs for a second tunnel crossing of Dry Creek in the Elverta Road Alternative
Right-of-Way	Baseline Road has a more usable right-of-way and would be more effective for construction. Homes and businesses on Elverta Road are very close to the current right-of-way and would create more difficulty during construction.
Traffic Impacts	Reported 24-hour traffic counts for Elverta Road were 18,548 vehicles, while 7-day traffic counts for Baseline Road was 13,657 vehicles. The traffic impacts would be roughly proportional to the traffic counts indicating that traffic impacts on Elverta Road would be more than 7 times greater that on Baseline Road.
Population	Population density on Baseline Road is much lower than along Elverta Road, the project would thereby have reduced impacts.
Disruption of Utilities	Utility disruption on Elverta Road has the potential to be more significant than for the Baseline Road route alternative based on preliminary observations
Public Concerns	Impacts to the public from construction on Baseline Road would be less significant than on Elverta Road, due to the larger number of homes and business owners along Elverta Road whose access to their properties could be affected, and who would be subject to dust, construction noise, and traffic reductions.

As a result of this preliminary analysis, it is recommended that the Baseline Road alternative be retained as the preferred pipeline route for the Elkhorn/Elverta Diversion alternative as defined in the Phase I Engineering Report.

APPENDIX D

WATT AVENUE VS.
WALERGA ROAD PIPELINE
PRELIMINARY ROUTING ANALYSIS

Draft Technical Memorandum

Elverta Diversion Alternative

Watt Avenue vs. Walerga Road Pipeline Preliminary Routing Analysis

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

The purpose of this Technical Memorandum (TM) is to evaluate two alternative routes for a segment of the pipeline to be installed as part of the proposed Elverta Diversion Alternative, as defined in the Phase I Engineering Report.[1] The overall pipeline would convey treated water from the Elverta Road Water Treatment Plant (WTP), located on Elverta Road near Garden Highway (**Figure 1**), to the Placer County Water Agency (PCWA), City of Roseville (Roseville), and Sacramento Suburban Water District (SSWD). The pipeline segment evaluated in this TM is the portion of pipeline between the intersection of Baseline Road and Watt Avenue and the intersection of Antelope Road and Walerga Road, (i.e., the pipe delivering water to SSWD).

PRELIMINARY ANALYSIS OF ROUTE ALTERNATIVE

The first alternative route would follow Watt Avenue from Baseline Road to Antelope Road, and Antelope Road from Watt Avenue to Walerga Road, where it would connect with the existing SSWD system (referred to as the Watt Avenue alternative). The second alternative would follow Baseline Road from Watt Avenue to Walerga Road and Walerga Road from Baseline Road to Antelope Road (referred to as the Walerga Road alternative; see **Figure 1**). The Watt Avenue route was chosen as the preferred route in the Phase 1 Engineering Report based on aerial photography, current street maps, existing information provided by the cost-sharing partners, and field investigation. The approach used to determine the preferred alignment included avoiding encroachment onto private property following the most direct route on roadways or existing rights-of-way (ROW), avoiding major disruption to existing utilities, and, where possible, avoiding highly populated areas. However, in Sacramento River Water Reliability Study (SRWRS) December and January Study Management Team meetings, a representative of PCWA suggested that the Walerga Road route might be preferable and requested that it be given further consideration. This route might be less expensive. The Walerga Road alternative included a shorter total length of pipe and the opportunity for the pipe to cross Dry Creek suspended from a planned new bridge rather than tunneling under the creek. To address this request, existing conditions along both routes were evaluated according to the criteria listed in **Table 1**.

Table 1. Route Analysis Criteria

Criteria	Description
Capital Cost	Total cost associated with design, construction, and labor
Right-of-Way (ROW)	Existing area available for construction, operation, and maintenance
Traffic Impacts	Traffic impacts expected during construction (e.g., road closures, lane reductions)
Population	Population density and distribution
Disruption of Utilities	Initial assessment of quantity and types of utilities expected to be encountered
Public Concerns	Factors that could result in negative impacts to the public (i.e., reduced access to residences or public facilities, noise, reduced access to business resulting in loss of income)
Biological Impacts	The potential to impact habitat for endangered species or species of concern.

[1] MWH. 2003. Appendix C, Phase I Engineering Report, Sacramento River Water Reliability Study. September.

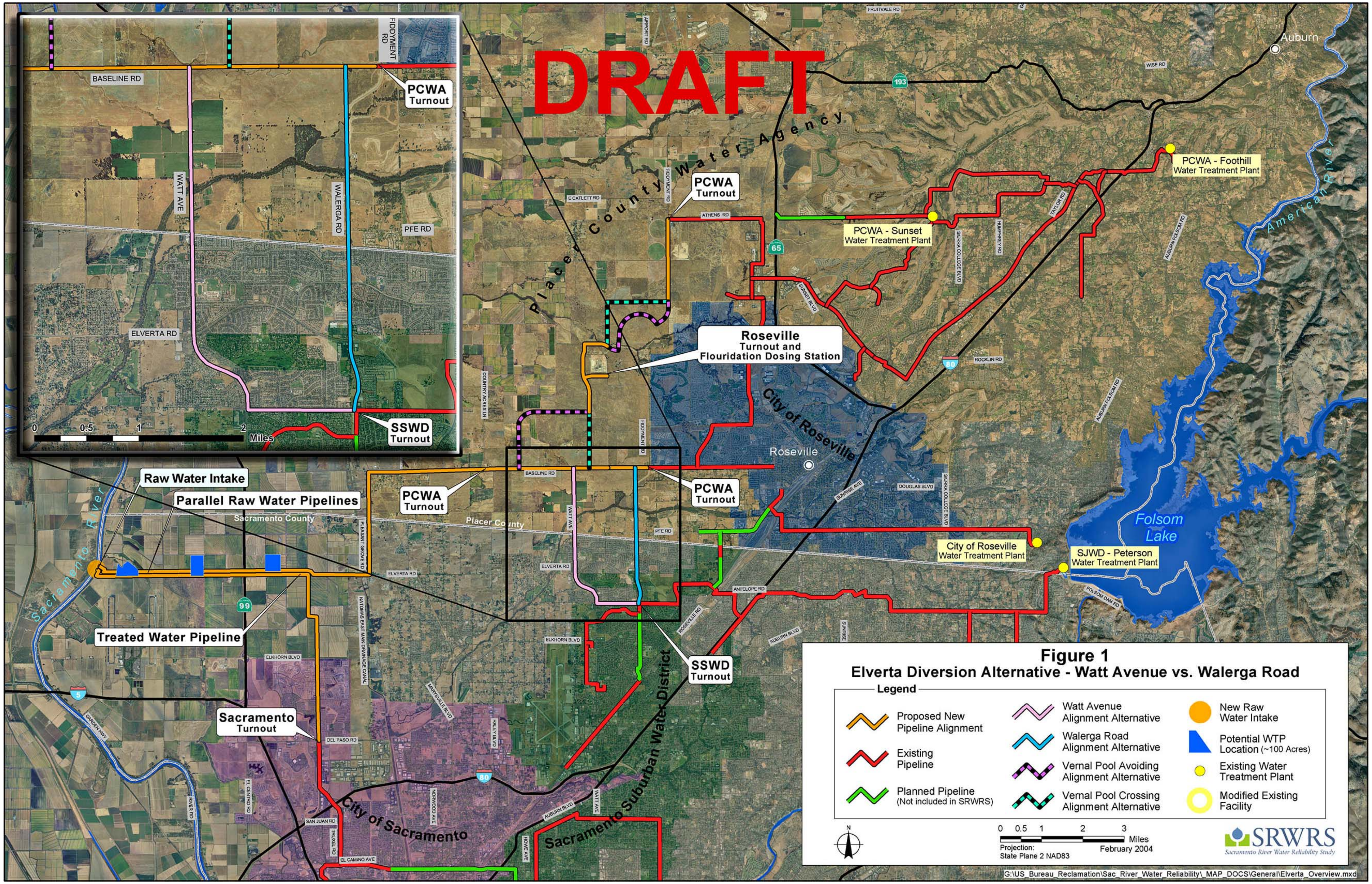


Figure 1 Elverta Diversion Alternative – Watt Avenue vs. Walerga Road

Watt Avenue Alternative

For the Watt Avenue alternative (**See Figure 1**), a 30-inch diameter pipeline would tee off of the pipeline in Baseline Road and follow Watt Avenue to its intersection with Antelope Road where it would turn east and follow Antelope Road to the intersection with Walerga Road. The pipeline would connect here to the existing SSWD distribution system. Downstream of the tee at Baseline Road and Watt Avenue, the pipeline in Baseline Road would be a 54-inch-diameter pipeline traveling east approximately 1,800 feet, then turning north to serve the City of Roseville and the PCWA Sunset area. An 18-inch pipeline would continue east on Baseline Road to the PCWA turnout at Baseline Road and Fiddyment Road. An evaluation of the Watt Avenue alternative with respect to the six route analysis criteria is presented in the following sections.

Capital Cost

This section develops cost estimates associated with installing the pipeline for the Watt Avenue alternative. Estimates include costs for materials, installation, and labor. Cost estimates are based on a unit cost, as discussed below.

Pipeline Unit Costs

To determine the cost of constructing the conveyance pipeline, a unit cost was developed for each pipeline condition. Each pipeline segment with similar characteristics was classified: Segment A – 54-inch pipe, normal traffic and utilities, Segment B – 18 inch pipe, normal traffic and utilities, Segment C – 30 inch pipe, normal traffic and utilities, and Segment D – 30 inch pipe, heavy traffic and utilities. **Figure 2** shows a schematic of the four segments of the Watt Avenue alternative and the length of each segment. The corresponding unit cost for each pipeline segment is as follows: normal traffic and utilities - $10/diameter-inch/linear food, and heavy traffic and utilities - $12/diameter-inch/linear foot. Estimates were considered accurate at the feasibility-level of the SRWRS, and may range between 30 percent above and 20 percent below actual construction costs.

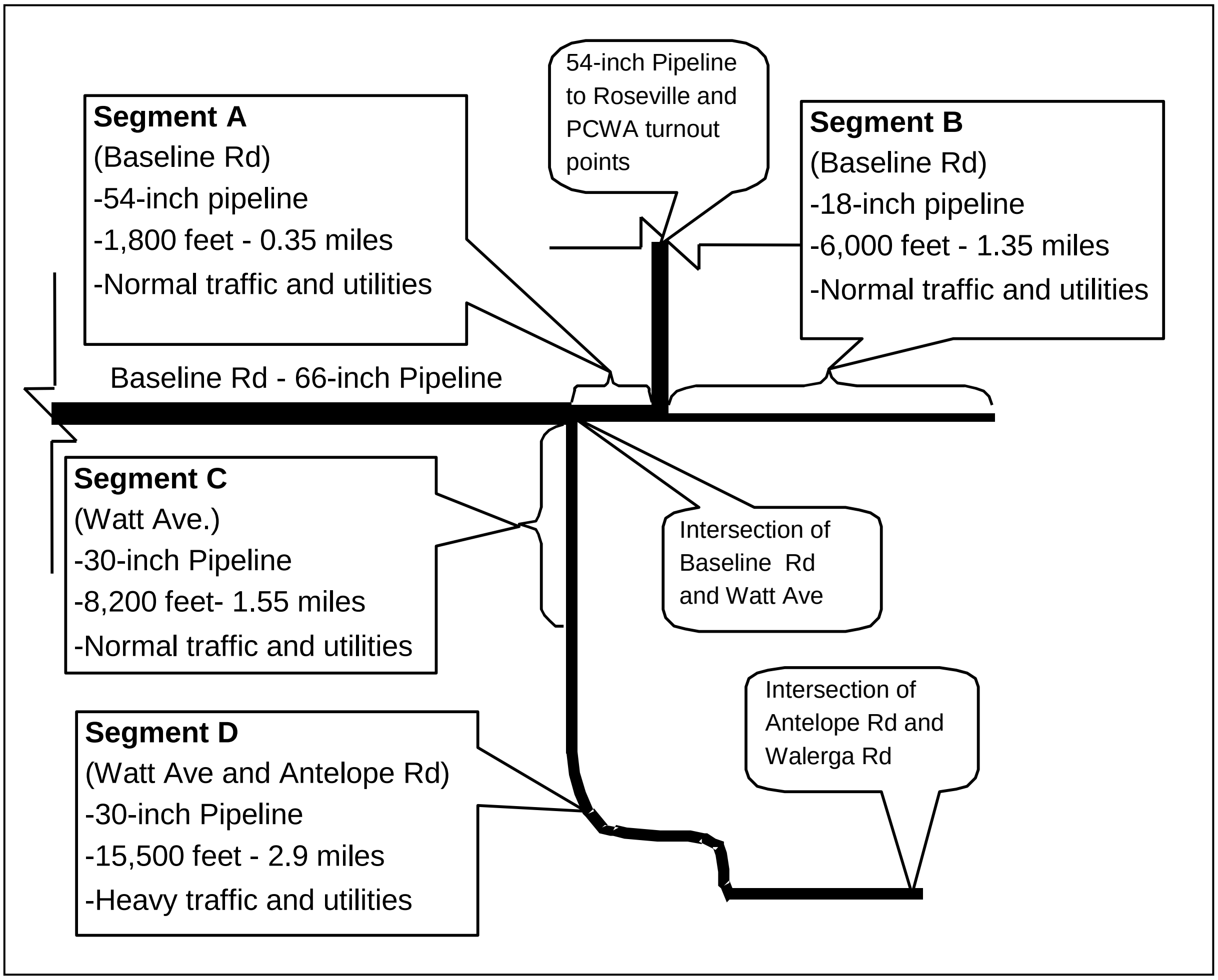


Figure 2. Segment Analysis for the Watt Avenue Alternative

Tunneling Costs

For the Watt Avenue alternative, the 30-inch pipeline would cross Dry Creek, a medium-sized stream. Therefore, in addition to costs for materials, costs would be incurred for tunneling. Tunneling costs were estimated based on other projects that involved creek crossings under similar conditions. Dry Creek is approximately 250 feet wide (bank to bank); a distance of 150 feet was added to this length for installing launching and receiving shafts set back beyond the bank (50-feet-deep shafts were assumed for this crossing). The tunneling cost used for a 30-inch pipeline is $27/diameter-inch/linear foot.

Estimated Route Cost

The total cost for the Watt Avenue Alternative is $16,549,000, as shown in **Table 2** below.

Table 2. Cost Estimate for Watt Avenue Alternative

Description	Unit	Quantity	Cost/Unit	Extended Cost
PIPELINE – BASELINE ROAD ALTERNATIVE				
Segment A – 54-inch diameter	lf	1,800	$540	$972,000
Segment B – 18-inch diameter	lf	6,000	$180	$1,080,000
Segment C – 30-inch diameter	lf	8,200	$300	$2,460,000
Segment D – 30-inch diameter	lf	15,500	$360	$5,580,000
Subtotal				**$10,092,000**
TUNNELING - DRY CREEK ON WATT AVENUE				
Tunneling – 30-inch Pipeline	lf	400	$810	$324,000
Tunneling Mobilization	ls	1	n/a	$100,000
Tunneling Shafts[1]	cy	3000	$25	$75,000
Subtotal				**$499,000**
Watt Avenue Alternative Cost				
Subtotal				**$10,591,000**
25% Engineering, Environmental, Administration and Legal Fees	25%			$2,648,000
Subtotal				**$13,239,000**
25% Contingency Fees	25%			$3,310.,000
TOTAL CAPITAL COST				**$16,549,000**

[1] Cost Includes launching and receiving Shafts

Key:
cy cubic yard
ft feet
lf linear foot
n/a not applicable

Right-of-Way

The ROW for Baseline Road between Watt Avenue and Fiddyment Road is 60 feet centered on the roadway centerline. The existing roadway is a two-lane paved road approximately 30 feet wide. The current ROW /roadway configuration provides good construction access. Placer County Road Expansion and Improvements plans to widen Baseline Road to a four-lane road; the timeline for this expansion has not yet been determined.

The ROW for Watt Avenue north of the Placer County/Sacramento County line (Segment C) is 40 feet wide. The roadway in Segment C is a 30-feet wide, two-lane road. Sufficient room in the roadway shoulder is avoidable for constructing the pipeline. Watt Avenue south of the county line (in Segment D) has four traffic lanes, a wide median, and wide, paved shoulders backed by gutters, curbs, and sidewalks. The pipeline would have to be constructed in the shoulder or one of the traffic lanes. This section of Antelope Road also has four traffic lanes with bike lanes and a paved shoulder on each side of the road backed by gutters, curbs, and sidewalks. The pipeline in Antelope would have to be constructed in the shoulder or one of the traffic lanes.

Traffic Impacts

Baseline Road is a moderate-to-high-use east/west corridor in Placer county. Traffic control measures would be required to maintain an adequate flow of traffic. Traffic control measures such as nighttime construction could be required and would likely result in slower production rates for open cut construction.

Watt Avenue north of the county line is a moderate-to-high-use north/south corridor in Placer County. Traffic control measures would be required to maintain an adequate flow of traffic. Traffic control measures such as nighttime construction could be required and would likely result in slower production rates for open cut construction.

Watt Avenue and Antelope Road are high-use traffic corridors. It is probable that the shoulder and one traffic lane would be closed during construction. Traffic control measures would be required to maintain safe traffic flow. Traffic would be slowed by the construction.

Population Density

Population density along Baseline Road and along Watt Avenue north of the county line is low,[2] as shown in **Figure 3**. The population to the south of Baseline Road is projected to increase through urban development, while land use to the north is currently zoned for agricultural use.

Population along Watt Avenue south of the county line and along Antelope Road is average suburban density. Residents would be impacted by noise, dust, and construction traffic.

Disruption of Utilities

The extent of domestic utilities serving communities in nearby areas can be estimated in **Figure 3**. During an initial site visit, few utilities were observed along Baseline Road. Watt Avenue and Antelope Road have the normal utilities encountered in urban streets.

Public Concerns

The key concern for the Watt Avenue alternative would be local and through traffic disruption. Construction noise and dust also would be of concern due to development along portions of this route.

Biological Impacts

The routes for the Watt Avenue and Walerga Road alternatives cross similar amounts of undeveloped and developed land. The potential for biological impacts is higher in the undeveloped areas. Since the amount of pipeline alignment in undeveloped areas is similar for the two alternatives, biological impacts are expected to be similar.

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[2] Population information gathered from Census 2000, Population Data

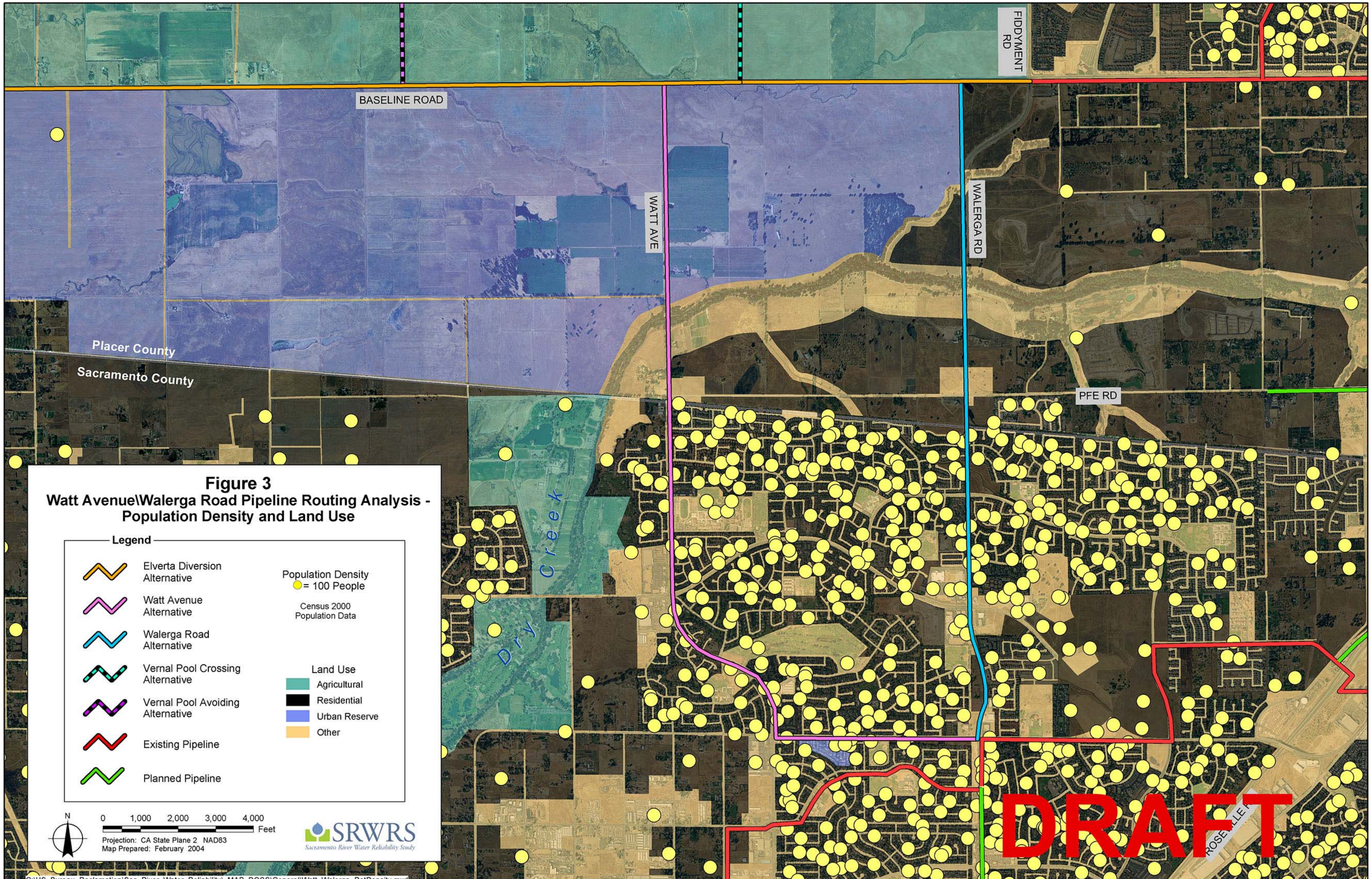

Figure 3 Watt Avenue/Walerga Road Pipeline Routing Analysis – Population Density and Land Use

Walerga Road Alternative

For the Walerga Road alternative (**See Figure 1**) a 66-inch diameter pipeline would follow Baseline Road from Watt Avenue eastward approximately 1,800 feet, where the 66-inch pipe would turn north to serve Roseville and PCWA. A 36-inch pipeline would continue east on Baseline Road to the point where Walerga Road formerly intersected with Baseline Road. At this point, the pipeline would branch, with an 18-inch pipe running eastward to the PCWA turnout at Baseline Road and Fiddyment Road and a 30-inch pipeline running south. The 30-inch pipe would follow Old Walerga Road and Walerga Road to its intersection with Antelope Road, where it would connect to the existing SSWD distribution system. An evaluation of the Walerga Road alternative with respect to the six route analysis criteria is presented in the following sections.

Capital Cost

This section develops cost estimates associated with installing the pipeline for the Walerga Road alternative. Estimates include costs for materials, installation, and labor. Cost estimates are based on unit costs, as discussed below.

Pipeline Unit Costs

To determine the cost of installing the conveyance pipeline, a unit cost was developed for each pipeline condition. Each pipeline segment with similar characteristics was classified: Segments A – 66 inch pipe, normal traffic and utilities, Segment B - 36 inch pipe, normal traffic and utilities, Segment C – 30 inch pipe, normal traffic and utilities, and Segment D – 30 inch pipe, heavy traffic and utilities. **Figure 4** shows a schematic of the four segments of the Walerga Road alternative and the length of each segment. The corresponding unit cost for each pipeline segment is as follows: normal traffic and utilities – $10/diameter-inch/LF, and heavy traffic and utilities – $12/diameter-inch/LF. Estimates were considered accurate at the feasibility-level of the SRWRS, and may range between 30 percent above and 20 percent below actual construction costs.

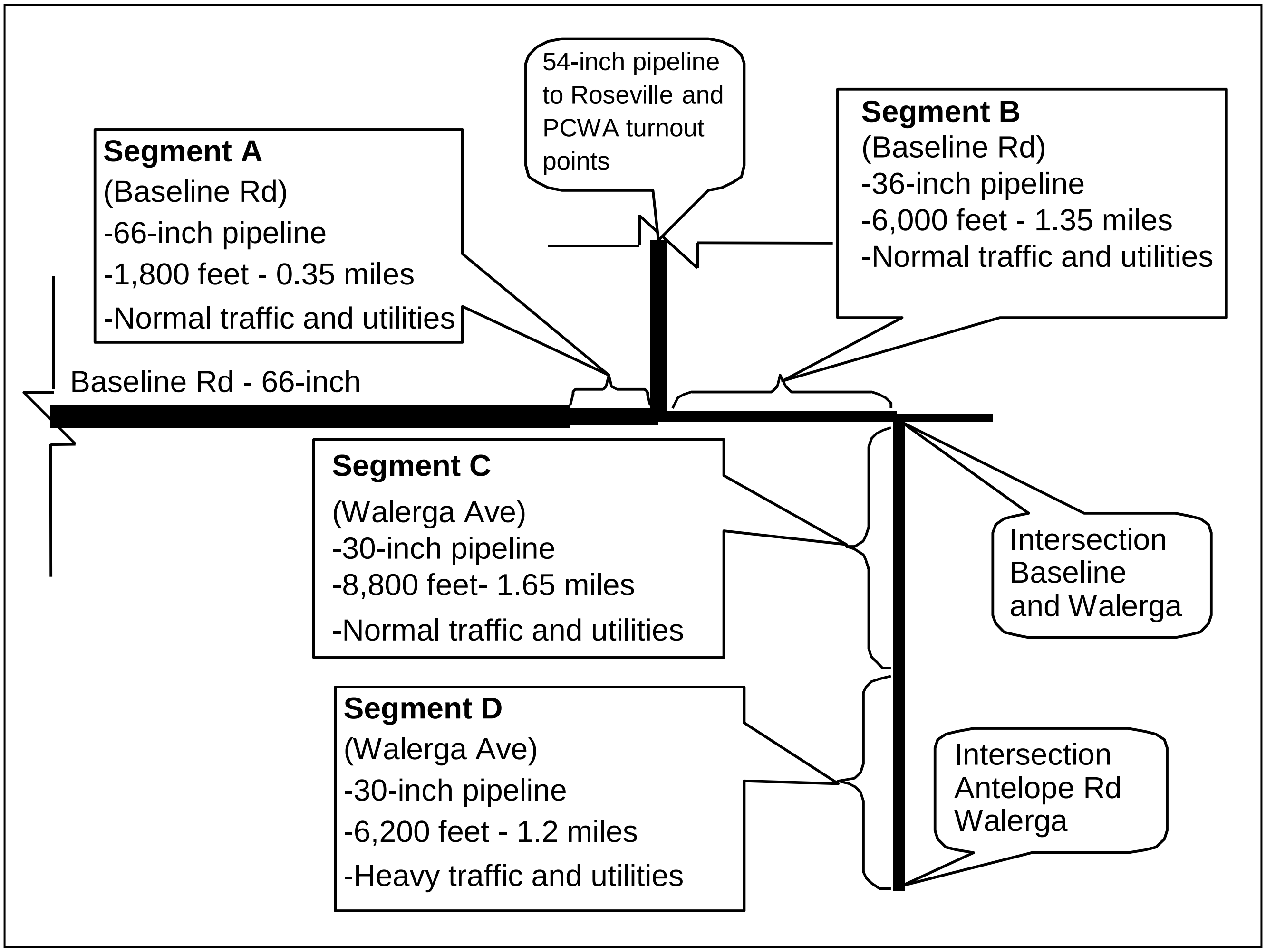

Figure 4. Segment Analysis for Walerga Road Alternative.

Bridge Crossing Costs

For the Walerga Road alternative, the 30-inch pipeline would cross Dry Creek at a location where Placer County is planning to construct a new bridge or widen the existing bridge. It is expected that environmental constraints would make it difficult to use open-cut trenching techniques to install the pipe across Dry Creek; however, it would be possible to construct or widen the bridge over Dry Creek in such a manner that the pipe could be attached to the side of the bridge. This would cause a nominal increase in the cost of the bridge, if pipe loads were taken into account during bridge design. If the pipe were attached to the bridge, it would mean that the pipeline serving SSWD could not be constructed until after the bridge was widened. Costs for constructing a pipe attached to a bridge are estimated to be $20 per inch-diameter per foot. Costs for constructing the pipe in the bridge approaches would be higher than for other portions of the road, since the work area would be narrower, so pipeline construction costs in the bridge approaches is estimated to cost $14 per inch-diameter per foot.

Estimated Route Cost

The total cost for the Walerga Road Alternative is $13,101,000 as shown in **Table 3**.

Table 3. Cost Estimate for Walerga Road Alternative

Description	Unit	Quantity	Cost/Unit	Extended Cost
PIPELINE - WALERGA ROAD ALTERNATIVE				
Segment A – 66-inch diameter	lf	1,800	$660	$1,188,000
Segment B – 36-inch diameter	lf	6,000	$360	$2,160,000
Segment C – 30-inch diameter	lf	8,800	$300	$2,640,000
Segment D – 30-inch diameter	lf	6,200	$360	$2,232,000
Subtotal				**$8,220,000**
BRIDGE CROSSING - DRY CREEK ON WALERGA ROAD				
Pipe on Bridge	lf	100	$600	$60,000
Pipe in Bridge Approaches	lf	250	$420	$105,000
Subtotal				**$165,000**
Walerga Road Alternative Cost Subtotal				$8,385,000
25% Engineering, Environmental, Administration and Legal Fees				$2,620,000
Subtotal				**$10,481,000**
25% Contingency Fees				$2,620,000
PIPELINE AND TUNNELING COST				**$13,101,000**

Key:
lf linear foot

Right-of-Way

The ROW for Baseline Road between Watt Avenue and Fiddyment Road is 60 feet centered on the roadway centerline. The existing roadway is a two-lane paved road approximately 30 feet wide. The current ROW/roadway configuration provides good construction access. Placer County Road Expansion and Improvements plans to widen Baseline Road to a four-lane road; the timeline for this expansion has not yet been determined.

Old Walerga Road is an abandoned section of Walerga Road running approximately 2,200 feet from Baseline Road south to Walerga Road. The ROW still belongs to Placer County, although plans for the ROW are uncertain. Construction of a pipeline in Old Walerga Road would be quite simple, with no traffic and few utilities. Should Placer County decide to sell this ROW, it would be possible to locate the pipeline in the new section of Walerga Road to the east of Old Walerga Road, although construction costs would be somewhat higher.

From the intersection of Old Walerga Road and Walerga Road south to the county line, Walerga Road is a two-lane roadway with unpaved shoulders. Sufficient room exists in the roadway shoulder for construction of the pipeline, although current and planned subdivision construction along this section of Walerga Road may increase the complexity of constructing a pipeline in the near future due to added curbs, gutters, utilities, and traffic.

South of the county line, Walerga Road is a four-lane road with bike lanes, paved shoulders, curbs, gutters, and sidewalks. The pipeline would need to be constructed in the shoulder and/or in one of the traffic lanes.

ROW concerns and complications are expected to be similar for the Watt Avenue and Walerga Road alternatives.

Traffic Impacts

Baseline Road is a moderate-to-high-use east/west corridor in Placer County. Traffic control measures would be required to maintain an adequate flow of traffic. Traffic control measures such as nighttime construction could be required and would likely result in slower production rates for open cut construction.

Walerga Road north of the county line is a moderate-to-high-use north/south corridor in Placer County. Traffic control measures would be required to maintain an adequate flow of traffic. Traffic control measures such as nighttime construction could be required and would likely result in slower production rates for open cut construction.

Walerga Road south of the county line is a high-use traffic corridor. It is probable that the shoulder and one traffic lane would be closed during construction. Traffic control measures would be required to maintain safe traffic flow. Traffic would be slowed by construction.

Traffic impacts would be slightly greater for the Watt Avenue alternative than for the Walerga Road alternative, since the Watt Avenue alternative would impact a greater length of heavy traffic streets.

Population Density

Population density along Baseline Road and along Walerga Road north of the county line is low[3], as shown in **Figure 3**. The population to the south of Baseline Road is projected to increase through urban development, while land use to the north is currently zoned for agricultural use. Some subdivision construction along Walerga Road in Placer County is currently underway.

Population along Walerga Road south of the county line is average suburban density. Residents would be impacted by noise, dust, and construction traffic.

Impacts on the resident population would be similar for the Watt Avenue and Walerga Road alternatives.

Disruption of Utilities

The extent of domestic utilities serving communities in nearby areas can be estimated in **Figure 3**. During an initial site visit, few utilities were observed along Baseline Road. Some utilities were apparent in Walerga Road in Placer County. Walerga Road in Sacramento County has the normal utilities encountered in urban streets. The Watt Avenue alternative would cause slightly more utility disruption than the Walerga Road alternative, because it has a greater pipeline length in developed areas.

Public Concerns

The key concern for the Walerga Road alternative, as with the Watt Avenue alternative, would be local and through traffic disruption. Construction noise and dust also would be of concern due to the development along portions of this route.

[3] Population Information gathered from Census 2000, Population Data.

CONCLUSION

The two route alternatives are compared in **Table 4** with respect to each of the six criteria used in this evaluation.

Table 4. Comparison of Alternatives

Cost	Estimated costs of construction for the Watt Avenue alternative ($16,549,000) are somewhat higher than the estimated costs of construction for the Walerga Road alternative ($13,101,000). The difference is due to less total length of pipeline and a cheaper crossing for Dry Creek.
Right-of-Way	Right-of-way concerns are similar for the two alternatives.
Traffic Impacts	Traffic impacts would be severe for both alternatives, but slightly worse for the Watt Avenue alternative as compared to the Walerga Road alternative.
Population	Population densities are similar for the two alternatives.
Disruption of Utilities	Utility disruption would be slightly greater for the Watt Avenue alternative, since more of its length is in developed streets.
Public Concerns	Impacts to the public from construction would be similar for the two alternatives.
Biological Impacts	Biological impacts are expected to be similar for the two alternatives.

Based on the findings of this preliminary analysis, it is recommended that the Walerga Road alternative be retained as the preferred pipeline route for the Elverta Diversion Alternative.